University of Alberta

Effectiveness Analysis of Different Bridge Deck Overlays in Alberta

By

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ABSTRACT

A bridge's deck deteriorates the fastest among all the bridge components and needs to be repaired, rehabilitated or even replaced after certain years of service. Since bridge deck overlays are one of the most common bridge deck rehabilitation methods used in Alberta, it was decided to investigate the effectiveness of different bridge deck rehabilitation overlay methods. This study reviews definition of five different bridge deck overlays, as well as their specification and mix design in Alberta. The effectiveness of these bridge deck overlays is discussed by service life prediction model and the result is compared with expert opinion method results. The impact of traffic and rehabilitation time on the service life is analyzed. The effectiveness of two main types of high performance bridge deck overlays, silica fume and fiber reinforced silica fume concrete overlays, is discussed. It is concluded that fiber enhances the overall performance of high performance concrete overlay.

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CHAPTER 1 INTRODUCTION

1.1 Background

Bridges play an important role in transportation systems. When bridges are damaged, the whole highway system will be paralyzed. Among all the bridge components, a bridge's deck deteriorates the most rapidly and requires rehabilitation or replacement before other components in that bridge deck are exposed to the traffic load and deicing salt directly (Bettigole, 1997).

Ahlskog (1990) has reported that the average service life of bridges in the USA is 70 years. Most of these bridges have reached half of their service life and need to be repaired immediately. In Alberta, a large amount of bridges were constructed during the 1950s or 1960s, and now these bridges are 40 to 50 years old and need major rehabilitation. Currently, bridge deck rehabilitation is very expensive. According to Alberta's 2006-09 fiscal plan, the entire budget for the provincial highway network is 3.6 billion dollars. The rehabilitation costs of deteriorated decks account for about one third to half of total bridge maintenance costs. As a result, the selection of appropriate bridge deck rehabilitation methods is a critical infrastructure issue.

Before selecting an appropriate bridge deck rehabilitation method, it is necessary to investigate the main mechanism that causes a bridge deck to deteriorate. Factors such as corrosion of reinforcement, traffic, moisture and temperature change can all cause a bridge deck to deteriorate. Deicing salt, which was applied since the 1960's in Alberta, makes chloride-induced reinforcement steel corrosion the major factor affecting bridge deck deterioration.

There are different methods to control reinforcement steel corrosion after patching or removing of damaged concrete. The principles of these rehabilitation methods for existing bridges are summarized by Virmani and Clemena (1998) and are listed below:

- Provide a barrier on the surface of the existing concrete to prevent ingress of chloride, for example overlays and membranes;
- Control the electrochemical reactions at the surface of reinforcement steel to decrease or stop metal loss, for example cathodic protection method;
- Electrochemical chloride extraction methods which could modify the concrete environment to make it less corrosive.

These three types of rehabilitation methods have proven effective in protecting a bridge deck. The bridge deck overlays method is an effective method used by Alberta Infrastructure and Transportation (AIT). Since the cost of using the overlay method for bridge deck rehabilitation is high, it is necessary to find out the most cost effective method.

1.2 Scope and Objectives

The main focus of this study is to evaluate the effectiveness of several bridge deck rehabilitation overlay methods in Alberta. In order to achieve this goal, specific objectives of this study are as follows:

- Evaluate the effectiveness of different bridge deck rehabilitation overlay methods by statistical analysis and expert opinion methods;
- Investigate the impact of combining fiber with high performance concrete overlay by comparing the performance of silica fume and fiber reinforced silica fume concrete bridge deck overlays;
- Determine the impact of traffic and overlay age on the effectiveness of bridge deck overlays;

1.3 Thesis Organization

In Chapter 2, a literature review is carried out to summarize various bridge deck performance test methods, especially the Copper-copper Sulphate Electrode (CSE) test. Three types of bridge deck service life prediction models are then reviewed including deterministic, stochastic and artificial intelligence models. Previous studies on the definition of bridge deck rehabilitation effectiveness and estimation of bridge deck overlay service life are described in this chapter.

Chapter 3 describes the bridge construction and rehabilitation history in Alberta. The definition of five different bridge deck overlays, along with their specification and mix design in Alberta, is also explored. Details of Earth Tech (ET) and AT (Alberta Transportation) bridge deck overlay service life prediction models are then explained.

Chapter 4 compares the effectiveness of five bridge deck rehabilitation overlay methods. AT and ET models are validated by comparing model prediction value with field test value. The service life of five different bridge deck overlays are estimated by both models, and a mean value of each group was calculated to compare the effectiveness. The impact of traffic and rehabilitation time on the service life is analyzed by using a regression analysis.

Since high performance bridge deck overlay is the main bridge deck rehabilitation method, and silica fume (SF) and fiber reinforced silica fume (FRSF) concrete overlay are the two main high performance bridge deck overlays, a decision was made to compare the effectiveness of SF and FRSF concrete overlay. Chapter 5 determines the performance indicators for these two types of deck overlays, which are crack length, crack density, rehabilitation service life and condition rating. A T-test will be used to determine the statistical difference between these two overlays when assumptions of T-test are met. Otherwise, a non-parametric test method will be used. The impact of traffic and overlay age on the effectiveness of bridge deck overlays will be investigated by using the regression analysis. Chapter 6 summarizes the finding of this thesis, and also provides an indication of possible trends for future research.

CHAPTER 2 LITERATURE REVIEW

It is important to understand the cause and extent of bridge deck damage before conducting bridge deck rehabilitation. Therefore, this chapter explains different types of bridge deck condition evaluation tests. In addition, this chapter examines different methodologies that have been used in predicting the service life of bridge deck overlay. The final section focuses on reviewing the previous studies, which estimated the service lives of different bridge deck rehabilitation overlay methods.

2.1 Bridge Deck Condition Evaluation Tests

Full bridge deck evaluation contains the following two-stages: preliminary survey and detailed survey. The preliminary survey is applied in order to determine the nature of a bridge deck problem. The detailed survey is conducted to confirm the cause of the problem and quantify the extent of the problem (Broomfield, 1997).

There are many bridge deck deterioration mechanisms including steel reinforcement corrosion, alkali-silica reactivity, freeze-thaw and plastic shrinkage. Steel reinforcement corrosion is the main concern of this study. Therefore, only techniques that can be applied to test a bridge deck's steel reinforcement corrosion are explained in this study. Table 2.1 presents different tests and methods for evaluation of steel reinforcement corrosion in concrete.

The purpose of visual inspection is to provide a general assessment of the nature and extent of a bridge deck problem. It is conducted by human eyes and brains. HWYCON, an expert system, was developed by the Strategic Highway Research Program (SHRP) to help on-site engineers identify possible causes of material-related distresses regarding bridge deck concrete. The accuracy of visual inspection is very subjective and highly depends on the skills of operative (Broomfield, 1997).

Table 2.1 Methods for Bridge Deck Condition Surveying

Methods	Detects	Use
Visual	Surface defects	General
Hammer/Chain	Delimitations	General
Cover meter	Rebar depth	General
Phenolphthalein	Carbonated depth	General
Chloride content	Chloride in concrete	Specialist
Half cell	Corrosion risk	Specialist
Linear polarization	Corrosion rate	Specialist
Resistivity	Concrete resistivity Corrosion risk	Specialist
Permeability	Diffusion rate	Specialist
Impact/Ultrasonics	Defects in concrete	Specialist
Petrography	Concrete condition, etc	Specialist
Radar/Radiography	Defects, steel location, condition	Specialist

The volume of reinforcement steel corrosion products is several times larger than that of steel. As a result, corrosion products build up tensile strength in the concrete and can cause fractures between the rebar and concrete. This kind of delamination can be detected by the hammer/chain drag test or other complicated tests such as radar, infrared, sonic and ultrasonic tests. When a bridge deck surface is hit by a hammer, or dragged through by a chain, a hollow sound indicates that cracks existence and delamination of concrete.

Half cell potential measurements can assess the possibility of corrosion in reinforced concrete structures. Copper-copper Sulphate Electrode (CSE) is a standard half cell potential test, which measures the potential as an indication of corrosion of steel reinforcement according to ASTM C876. Interpretation of CSE test reading is listed in Table 2.2.

Table 2.2 Interpretation of CSE Test Readings (ASTM C876)

Half-cell potential reading	**Probability of Corrosion**
Less negative than -200 mV	>90% probability of no corrosion occurring
Between -200 mV and -350 mV	uncertain
More negative than -350 mV	> 90% probability of corrosion occurring

The impact of various factors on the reliability of the CSE test results has been explained by Gu and Beaudoin (1998). A dense concrete cover can limit the oxygen diffusion process. As a result, the corrosion potential will shift to a more negative value, which can not be used to indicate a high probability of steel corrosion. A high concrete resistance will introduce significant errors in the half-cell potential data. A decrease in oxygen concentration and concrete carbonation will result in a shift towards more negative values of half-cell potential readings. The influence of a corrosion inhibitor on reading results could be in either direction depending on anodic or cathodic inhibitor.

In general, the half-cell potential measurement only revealed the corrosion probability at a given location and time, and there are many combinations of factors affecting the precision of half-cell potential readings. In order to correctly evaluate the degree of rebar corrosion, many factors must be taken into account and long-term monitoring of the half-cell potential reading will make the technique more meaningful (Gu and Beaudoin, 1998).

Chloride content inside concrete may be tested in a laboratory or in the field. Concrete samples are drilled from the field concrete and dissolved in the acid or water and titrated to find the chloride concentration. Quantab strips and specific ion electrodes are the main two field chloride content test methods. It is expensive to conduct field tests, due to the costly equipment, and the test results must be validated by laboratory test results (Broomfield, 1997).

The chloride content at the rebar indicates the current extent of corrosion activities. When the chloride content exceeds one threshold value, approximately 0.2 to 0.4% chlorides by weight of cement, or 0.05% by weight of concrete, the passive layer of reinforcement steel will break down due to the presence of chloride (Broomfield, 1997).

Other Tests

The Galvanostatic pulse method is a rapid non-destructive polarization technique. It has been successfully applied on structures in wet and anaerobic environments, and where half-cell potential technique is difficult to implement (Klinghoffer, 1995).

Ground Penetration Radar (GPR) method is another non-destructive evaluation method where pulses of microwave energy are directed at, and reflected from, the various layer interfaces in the deck system. Barnes and Trottier (2004) collected data from 92 decks in Nova Scotia, from 1996 to 2000, using the GPR test. At the same time, ground-truth data, namely field test data, were obtained using the chain drag and half cell potential survey methods on 24 of the 92 GPR-surveyed decks. For the deteriorations that were less than 10% of the surface area, according to the chain drag survey, a moderate to highly significant difference was observed between GPR versus ground-truth method. For the deteriorations that were above 50% of the surface area, the difference was considered moderate to high. GPR surveys between 10% and 50% deterioration, according to ground-truth method survey, had a relatively high accuracy in predicating the repair. However, GPR was still ineffective at mapping out detailed locations of deterioration on the deck. Using a combination of GPR and visual inspection methods could be an effective way of mitigating the gross difference between estimated and measured deck repair quantities and the resulting high cost overruns.

2.2 Service Life Prediction Models

Bridge engineers have been developing bridge deck service life prediction models for many years. A service life prediction model can not only estimate a bridge deck's remaining service life, and help engineers make decision about when and where to undertake the rehabilitation actions, but it can also evaluate the effectiveness of bridge deck overlays (Kirkpatrick, 2002).

Typically, the three types of service life prediction methods are as follows: deterministic, stochastic and artificial intelligence models. Table 2.3 lists all the categories of bridge deck service life prediction models.

Table 2.3 Categories of Bridge Deck Service Life Prediction Models

Category	Technique	Methods
Deterministic models	Straight-line extrapolation	-
	Regression models	Stepwise regression
		Linear regression
		Non-linear regression
	Curve-fitting models	B-spline approximation
		Constrained least squares
Stochastic models	Simulation models	-
	Markovian models	Percentage prediction
		Expected-value method
		Poisson distribution
		Negative-binomial model
		Ordered-probit model
		Random-effects model
		Latent Markov-decision process
Artificial intelligence models	Artificial neural networks	-
	Case-based reasoning	-
	Machine learning	-

2.2.1 Deterministic model

Deterministic models are causal models and are expressed as the relationship between factors affecting bridge deck deterioration and a measure of bridge condition (Morcous, 2000). Depending on the mathematical and statistical methods used, the three types of deterministic models are as follows: straight-line extrapolation, regression models and curve-fitting models.

The straight-line extrapolation model assumes that a bridge deck will deteriorate at the same rate as in the past. Therefore, the future condition of a bridge deck can be predicted by extrapolating the previous two condition rating data (Morcous, 2000). This is a very simple method and is easy to use. However, it is not applicable when any of the impacting factors change or a bridge deck is rehabilitated.

Regression models establish the relationship between the condition rating of a bridge deck and one or several impact factors. These factors include traffic, bridge structure type, bridge deck type, bridge deck age and the amount of salt applied. The most commonly used regression models are stepwise regression, linear regression and non-linear regression.

Before applying regression models to estimate future bridge deck conditions, bridges should be categorized into several groups based on the impact of factors in order to assure the homogeneity among bridges of the same group (Morcous, 2000). The formula of linear relationship is given by Shahin (1994) as:

$$Y_i = \beta_0 + \beta_1 X_i + \varepsilon_i$$

Where,

Y_i: Condition rating of a bridge i

X_i: Age of a bridge i

ε_i: Prediction error of a bridge i

β_0, β_1 : Regression parameter

If the linear regression model does not fit the data groups, a multiple linear regression model, a non-linear regression model and a curve fitting technique can be applied.

2.2.2 Stochastic model

Stochastic modes, unlike deterministic models, are capable of predicting the performance of bridge decks under a range of input values or conditions. They are widely used in modeling the infrastructure deterioration process. There are three types of stochastic models including probability distribution, Markov chain models and simulation techniques. Probability distribution model is suitable for project level bridge deck condition prediction. This model associates bridge deck condition rating values with all the probabilities (Shahin, 1994).

The Markov Chain model is the most popular stochastic bridge deck deterioration model. This method divides bridge deck conditions into several independent condition states and assumes that past condition states have no impact on the future bridge deck states. This method also assumes that the probability transition matrix is constant throughout certain prediction periods. The future bridge deck condition is calculated by multiplying the current bridge deck condition with a transition probability matrix. Many researchers have tried various methods to predict transition probability matrix, as are listed in Table 2.4.

Table 2.4 Transition Probability Matrix Estimation Methods

Methods	Authors
Percentage Prediction Estimation	Jiang et al. 1988
Expected Value Method	Madanat et al. 1995
Poisson's Regression and Negative Binomial Models	Madanat and Ibrahim 1995
Ordered Probit Model	Madanat et al. 1995
Random-Effect Model	Madanat et al. 1997

Kirkpatrick et al. (2002) adopted the Monte Carlo simulation method to predict a bridge deck's service life. The service life model for the chloride- induced corrosion of reinforcing steel in concrete involved an initiation and propagation period. The end of functional service life was defined to have been reached when 12 percent of the worst span lane of a bridge deck had deteriorated. The time for the first repair was reached when 2.5 percent of the worst span lane of a bridge deck had deteriorated. Two sampling techniques, parametric and simple, were implemented and the results were compared. Data were collected from ten bridge decks in Virginia, including cover depth, chloride content on powdered samples removed from the deck, surface chloride concentration and apparent diffusion coefficient. The service life prediction results from these two sampling methods matched very well, which indicated that the shape of the distribution of the input variables did not seriously affect the predicted time for first repair and rehabilitation. The accuracy of the predictions for the first repair and rehabilitation would be limited until the value or distribution of the chloride initiation concentration was better defined. The time for corrosion deterioration, to the end of functional service, was still not known.

2.2.3 Artificial intelligence models

A multi-layer Artificial Neural network (ANN) was used by Sobanjo (1997) to predict the bridge superstructure condition rating by relating it with bridge age. The ANN had one input layer, two hidden layers and one output layer. The input was the bridge's age and the output was a corresponding condition rating. Fifty bridges were used for network training. The result showed that 79% of the predicted value matched the measured value with a prediction error of less than 15%.

Morcous et al. (2002) applied the case based reasoning (CBR) method in bridge deck service life prediction. The CBR method solved the current problem by reusing previous cases that were similar to the current problem. The assumption of the CBR method was that two bridges would have similar performances if they were in similar environmental or operational conditions and had similar inspection and maintenance history. Five-hundred and twenty-one bridge decks were used in data analysis, out of which 259 bridge decks were used for data validation. About 60-90 percentages of data retrieved correct solutions from the previous cases and gave correct predictions.

Melhem and Cheng (2003) attempted to use the machine learning (ML) method to predict the remaining service life of a bridge deck. There were two kinds of ML methods introduced in their paper, the k-nearest-neighbor instance-based learning (IBL) method and the inductive learning (IL) method. In the experiment, data were manually gathered from hard-copy form reports from the Kansas Department of Transportation bridge deck surveys. The bridge chosen had similar traffic volume and structural practice. Only the material and design factors were considered as the most pertinent to concrete bridge deck deterioration. The highest accuracy of predicating service life of a bridge deck was 41.8% using the IL model and 50% using the IBL model. Both these two numbers were typically considered low accuracies in classification and ML systems because the ML method could reach to the accuracy of 80% or better. It was feasible to use the IBL or the IL technique for engineering application in classification and prediction problems, such as estimating the remaining service life of bridge decks. Although these two methods had the disadvantage of being time consuming, the accuracy was higher than other predication methods. Between these two, the IBL was more efficient than IL for this application.

2.3 Service Life of Bridge Deck Rehabilitation Overlay Methods

In 1993, Strategic Highway Research Program (SHRP) published a report on concrete bridge protection, repair and rehabilitation. Bridge deck overlays were considered as both bridge deck repair and rehabilitation methods. Bridge repair methods (Weyers et al., 1993) were defined as "bridge components that have been patched and overlaid or encased and in which the sound, chloride-contaminated concrete has been left in place." Bridge rehabilitation methods were defined as bridge elements "where the spalled areas have been patched, the delaminated areas and all areas with a corrosion potential more negative than 250 mV to the copper/copper sulfate half cell have been removed and patched, and the entire surface has been overlaid or encased with low-permeability concrete." Service life of bridge deck repair overlays, listed in Table 2.5, was different from bridge deck rehabilitation overlays as reported in SHRP-S-30.

Table 2.5 Service Life of Repair Overlays (Weyers et al., 1993)

Overlay Types	**Service Life (years)**
Low-slump high density concrete overlay	22-26
Latex modified concrete overlay	22-26
Hot-mix asphalt with a preformed membrane	10-15

From Table 2.5, it was concluded that low-slump high density concrete overlay (LSDC) and latex modified concrete overlay (LMC) had longer service life than hot-mix asphalt with a preformed membrane (HMAM).

Rehabilitated bridge deck overlays had longer service life than repaired bridge deck overlays in that chloride-contaminated concrete was removed before the overlay was placed on top of it. Service life of rehabilitated bridge deck overlays was determined by the type of overlay, rate of chloride diffusion and environmental conditions. Therefore, service life of these three types of bridge deck overlays were categorized into four groups based on chloride environment condition, which were classified into low, moderate, high and severe level with chloride diffusion concentration of 1.2, 3.6, 5.3 and 7.4 kg/m3 respectively (Weyers et al., 1993). The service life of bridge deck overlays in each chloride condition is listed in Table 2.6.

Table 2.6 Service Life of Rehabilitation Overlays (Weyers et al., 1993)

Overlay Types	Low (years)	Moderate (years)	High (years)	Severe (years)
Low-slump high density concrete overlay	>100	35-70	30-60	25-50
Latex modified concrete overlay	>90	20-35	15-30	15-25
Hot-mix asphalt with a preformed membrane	30	30	25	20

While low-slump high density and latex modified concrete overlay showed the highest service lives, hot-mix asphalt, with a preformed membrane, showed the shortest service life.

In some cases, bridge decks need to be repaired in a very short time in order to minimize traffic delays and user costs. In 1993, SHRP published a report titled "Rapid Concrete Bridge Protection, Repair and Rehabilitation." Sprinkel et al. (1993) showed detailed flow charts for rapid bridge deck treatment in report SHRP-S-344. To be qualified as rapid rehabilitation, a bridge deck must be treated by one of the following four treatment methods within lane closure time conditions constrain of < 56, < 21, < 12 and < 8 hours.

- < 56 hour – semi rapid (e.g., Friday, 9 pm to Monday, 5 am)
- < 21 hour – rapid (e.g., 6 pm to 3 pm)
- < 12 hour – very rapid (e.g., 6 pm to 6 am)
- < 8 hour – most rapid (e.g., 9 pm to 5 am)

Literature review and expert opinion gave a close estimate of service life for all bridge deck overlays, as listed in Table 2.7. Normal Portland cement concrete overlay had the longest service life, with the average estimated service life around 15 years. Polymer overlay had a moderate service life, while asphalt concrete overlay on membrane had the shortest service life.

Table 2.7 Service Life of Rapid Bridge Deck Protection and Rehabilitation Treatments (Sprinkel et al., 1993)

Treatment	**Service Life Prediction Based on**					
	Questionnaire response			**Literature Review**		
	Avg.	**Low**	**High**	**Avg.**	**Low**	**High**
Asphalt concrete overlay on membrane	11.8	4.5	20	9.7	3.7	15
Portland cement concrete overlay	15.5	10	22.5	17.9	13.6	25
Polymer overlay	12.7	6	25	10	-	-

Service life estimations for rapid deck protection and rehabilitation treatments, based on field data, are listed in Table 2.8. The life of overlays was influenced by permeability of chloride icon, adhesion, and corrosion induced spalling and traffic (Sprinkel et al., 1993).

The projected minimum service life was for bridges in low traffic and not seriously contaminated concrete conditions (Sprinkel et al., 1993). The highest minimum service life is 25 years for multiple-layer epoxy overlay, multiple-layer epoxy-urethane overlay, premixed polyester overlay, latex and type III cement overlay and silica fume overlay.

In addition to the survey of service life, the National Research Program SHRP-S-344 also estimated the labour and material cost and construction time for high early strength PCC overlays and polymer overlays.

Table 2.9 Construction Hour and Cost (Sprinkel et al., 1993)

	Construction Hour (hours)	**Labor and Material Cost($/m^2)**
High early strength PCC overlay	18 - 36	110
Polymer overlay	-	38

Table 2.8 Years of Service life of Rapid Protection Treatments based on Field Evaluations (Sprinkel et al., 1993)

Protection Treatment (Overlays)	Age of Oldest Installation (years)	Projected Minimum Service Life (ADT 5000) (years)	Property Controlling Service Life	Adjustments for Traffic							
				Average Service Life (years)				Average Permeability Coulombs for ADT[1]			
				L	M	H	VH	L	M	H	VH
Multiple-Layer Epoxy	15	25	Permeability	0	0	-10	-15	125	300	700	800
Multiple-Layer Epoxy-Urethane	7	25	Skid Number	0	-10	-	-	125	150	-	-
Premixed Polyester	8	25	Adhesion	0	-	-	-	150	-	-	-
Methacrylate Slurry	6	18	Skid Number	0	-11	-13	-15	0	0	0	0
Multiple-Layer Polyester	9	10	Adhesion	0	0	0	0	1100	1250	1300	1350
Multiple-Layer Mechacrylate	9	15	Adhesion	0	0	0	0	1100	1250	1300	1350
Special Blended Cement	1	-	CIS[3]	-	-	-	-	-	-	-	-
Latex & Type III Cement	5	25	CIS	0	0	0	0	600	600	600	600
Silica Fume	5	25	CIS	0	0	0	0	600	600	600	600

1. ADT (Average daily traffic) where L is < 5,000; M is 5,000 to 25,000; H is > 25,000; and VH is > 50,000

2. CIS indicates corrosion induced spalling

Detwiler et al. (1997) compared the effectiveness of silica fume concrete overlay with high density low slump concrete overlay by using a unique case study. The bridge was located over Interstate 55 in Illinois, USA. The southbound bridge lane was repaired by a dense concrete mixture in October 1986, while the northbound bridge lane was repaired by silica fume concrete in March 1987. Both directions had the same traffic conditions, freeze-thaw cycle, and an amount of de-icing salt. The construction was completed by the same contractor. As a result, both sides had the same quality of work. The mix proportions of these two overlays are listed in Table 2.10.

Table 2.10 Mix Proportions and Properties for Overlay Concrete (Detwiler et al., 1997)

	Silica Fume Concrete	**Dense Concrete**
Water (kg/m^3)	129	157
Cement (kg/m^3)	327	488
Silica Fume (kg/m^3)	42	0
Coarse aggregate (kg/m^3)	943	824
Fine aggregate (kg/m^3)	806	837
Compressive Strength,7 days	40	39
14 days (MPa)	48	44
Air Content (%)	6.0 - 8.4	5.5 - 6.3

In 1995, a series of surveys were carried out including field examinations, petrographic examinations and chloride ion penetration tests. The field examination results indicated that both overlays had good performance. There was no surface scaling, but there was a good bond strength to the substrate and very few cracks. A petrography examination was inspected on several cores of each concrete overlay, according to the standard ASTM C 856-83. The results showed that both overlays were well consolidated and tightly bonded to the substrate. The chloride test results indicated that silica fume concrete overlay had lower penetrability than high density low slump concrete overlay. Since there was no chloride contents found in samples 30 mm lower than surface, the diffusion rate could be calculated from a chloride profile. There was lower chloride ion concentration in silica fumes concrete at the same distance from the surface. In general, both concrete overlays had a good performance after nine years of service. However, silica fume concrete overlay performed better with respect to chloride penetration.

In a survey conducted by Ramcharitar (2004), a questionnaire was distributed to numerous Departments of Transportation, municipalities, bridge authorities and consultants all over the USA and Canada. The results showed that five different types of bridge deck overlays, as listed in Table 2.11, performed well in both countries, though the effectiveness was a little better in USA. The effectiveness was defined by Ramcharitar as an indication of how effective these overlay methods had been for rehabilitation programs from the expert's experience.

Table 2.11 Effectiveness of Different Types of Overlays (Ramcharitar, 2004)

Concrete Replacement Overlays	**Range of Effectiveness (%)**		**Mean Effectiveness (%)**	
	Canada	**USA**	**Canada**	**USA**
Normal concrete	70-100	90-95	88	93
High performance concrete	70-90	75-100	81	92
High early strength concrete	75	70-90	75	80
Latex modified concrete	80-90	90-100	84	93
Asphalt concrete	80	70-100	80	88

High performance concrete overlay had an average effectiveness of 81% in Canada, while the USA had an average effectiveness of 92%. Asphalt concrete overlay had an average effectiveness of 80% in Canada and 88% in USA. The sequence of these five groups, by effectiveness, was as follows: normal concrete, latex modified concrete, high performance concrete, asphalt concrete and high early strength concrete.

A questionnaire was conducted to compare the effectiveness of different bridge deck overlays by Russell et al. (2004). Questions were asked regarding which overlay systems each transportation agency had used in the past and which they were currently using, along with the performance rating. The result is shown in Table 2.12.

Table 2.12 Use of Overlay Systems (Russell et al., 2004)

Overlay	No. of Respondents[1]		Performance Rating[2]	
	Past	Current	Range	Average
None	6	5	—	—
Asphalt without Membrane	28	16	1 to 5	3.6
Latex-Modified Concrete	26	20	1 to 5	2.4
Low-Slump Dense Concrete	26	12	1 to 5	2.4
Fly Ash Concrete	4	11	2 to 4	2.4
Silica Fume Concrete	10	21	1 to 3	2.0
Epoxy	11	11	1 to 5	2.6
Polyester	4	2	1 to 5	2.5
Other	5	4	1 to 5	2.8

[1] Total number of survey respondents = 45.

[2] 1= excellent, 5 = poor, — = not applicable.

The study concluded that asphalt, with a rating of 3.6, had the best performance. Epoxy and polymer had moderate performances with a rating of 2.6 and 2.5 respectively. Latex-modified concrete, low-slump dense concrete overlays and fly ash overlay had the lowest performances with each of them receiving a rating of 2.4.

In the recent study conducted by Huang (2004), a project-level decision support tool was developed that could rank maintenance scenarios for concrete bridge deck deterioration. Data were provided by the Wisconsin Department of Transportation, including service life of each maintenance scenario, cost range and productivity of each treatment, as shown in Table 2.13. It was summarized that concrete overlay has a higher service life than asphalt concrete overlay. Membrane could improve the service life of asphalt concrete overlay by 4-5 years. However, concrete overlay was twice as expensive as asphalt concrete overlay with membrane and four times expensive than asphalt concrete overlay without membrane. From an economic perspective, asphalt concrete overlay with membrane is the best choice because it has the highest cost service life ratio.

Table 2.13 Estimated Costs and Productivity of Concrete Deck Treatments (Huang, 2004)

Treatment	Estimated Service Life (Years)	District 1		District 5		Includes Traffic Control	Include Concrete Preparation
		Cost ($/m^2)	Productivity	Cost ($/m^2)	Productivity		
Asphalt Concrete Overlay without Membrane	2-3	-	-	53	-	Yes	Yes
		17-20	557 m^2/day	-	-	No	Yes
Asphalt Concrete Overlay with Membrane	7	-	-	53-107	-	Yes	Yes
		22	557 m^2/day	-	-	No	Yes
Concrete Overlay	15-20	290-334	1/3 bridge/week	107-216	278 m^2/30days	Yes	Yes
		161-216	1/3 bridge/week	-	558 m^2 /35-40days	No	Yes

CHAPTER 3 BIDGE DECK OVERLAYS IN ALBERTA

The first two sections of this chapter introduce bridge construction and rehabilitation history in Alberta. The following section discusses the definition, advantage and limitation of five different types of bridge deck overlay rehabilitation methods, along with their specification, mix design and applications in Alberta. The last section presents existing bridge deck service life prediction models used in Alberta.

3.1 Bridge Deck Construction and Rehabilitation History in Alberta

Bare bridge deck was widely used by Alberta Infrastructure and Transportation (AIT) for new bridge deck construction in the 1950's and 60's. Most big bridges, at that time, were constructed with cast-in-place decks on top of steel superstructure. Some rural bridges used precast concrete bridge girders as bridge deck wearing/riding surface. However, the application of de-icing salt during the 1970's caused a serious reinforcement steel corrosion problem in the old bare bridge deck, which resulted in revisions to bridge deck construction. In the 1970s, in order to prevent bridge decks from corrosion, some new bridges were constructed with different kinds of experimental membranes. A two-stage construction using High Density Concrete Overlay (HDOL) became the standard for constructing new bridges that may be exposed to de-icing salt. Epoxy coated rebar was also applied in bridge deck construction, in addition to HDOL in the 1980s. In 1985, a two-layer, 80-mm asphalt mixture was used for bridge deck construction when new riding surface standards changed from HDOL to hot rubberized membrane/protection board.

In the late 1960's, bridge decks were repaired primarily because of a chloride reinforcement corrosion problem. AIT placed concrete overlay on top of the old bridge deck and sometimes reinforced it with a welded wire mesh. These bridge deck overlays had poor performance with service life of approximately ten years. In 1976, AIT applied HDOL as an experimental bridge deck repair method on the Cushing Street Bridge in Calgary, Canada, and on the Little Smoky River Bridge at Guy in 1977. AIT then adopted HDOL as standard bridge deck repair methods with the goal of developing a bridge deck service life of 20 years. However, HDOL showed serious crack problems after several years of implementation. To solve this problem, AIT used High Performance Concrete Overlay (HPCO) as the new bridge deck repair method in the 1980's.

Table 3.1 summarized bridge deck overlay rehabilitation methods in Alberta, as well as years placed. In total there were 470 bridge decks rehabilitated by overlays, out of which 80 bridges were repaired with conventional reinforced overlay, 150 bridges were repaired with HDOL and 150 bridges were repaired with HPCO silica fume with steel fiber.

Table 3.1 Bridge Deck Overlay Rehabilitation Methods in Alberta

Overlay Type	Placed by	Years Placed	Approx Total
Class Density Concrete	AIT bridge crew	1968-1975	10
Deep Conventionally Reinforced	AIT bridge crew	1973-2005	80
Iowa High Density	Public Contract	1977-1985	150
Latex Modified Concrete	AIT bridge crew	1980-1993	5
Cathodic Protection Overlay	Both types	1982-1985	5
Class D W/Steel Fiber	AIT bridge crew	1984-1986	10
HPC-Silica Fume, Steel Fiber	AIT bridge crew	1987-2005	150
HPC-Pyrament, Steel fiber	AIT bridge crew	1989-1991	20
HPC-Silica Fume	Both types	1994-2005	30
HPC-Silica Fume, Fly Ash	Public Contract	2000-2005	10
Total			470

Note: HPC is abbreviation of high performance concrete

3.2 Bridge Deck Overlays in Alberta

3.2.1 High performance concrete overlay (HPCO)

High performance concrete (HPC) is defined by the American Concrete Institute as the "concrete meeting special performance and uniformity requirements that cannot be achieved routinely by conventional materials and normal mixing, placing and curing practices. The major characteristics of HPC are easy placement, high early-age strength, toughness, superior long-term mechanical properties and prolonged life in severe environment (ACI 1996). In Alberta, HPCO has been implemented in bridge deck rehabilitation since the 1980s. There are many different types of HPCOs depending on the types of admixtures added. Silica fume and fiber reinforced silica fume concrete overlays are the two most widely used HPCOs in Alberta. The properties of these two concrete overlays will be introduced in the following sections.

3.2.1.1 High Performance Concrete Overlay with Silica Fume

Silica fume (SF) concrete overlay was first adopted in bridge deck overlay rehabilitation in the 1970s. Silica is a byproduct of the reduction of high-purity quartz with coal in electric furnaces in the production of silicon and ferrosilicon alloys (FHWA Materials Group 1999). Silica fume can improve concrete properties by utilizing a micro-filling effect. The small particles of silica fume reduce or eliminate capillary channels, resulting in very little bleeding of concrete and improves chloride impermeability. However, plastic shrinkage cracking was the main problem in SF concrete overlays.

3.2.1.2 Fiber Reinforced Silica Fume Concrete Overlay

Steel fiber reinforced concrete is conventional Portland cement concrete mixed with discontinuous discrete steel fibers. Steel fiber reinforced silica fume (FRSF) HPCO is the most widely used overlay rehabilitation method in Alberta.

The main characteristic of FRSF concrete is its toughness. Silica fume creates more densified mixtures than traditional Portland cement concrete, and as a result, improves chloride impermeability. Steel fiber has elastic modulus 10 times stronger than conventional concrete cement. It can arrest the prorogating micro-crack and distribute stress homogeneously throughout the mix. Therefore, steel fiber can significantly improve the compression, tension and flexure properties of conventional concrete. In particular, steel fiber can greatly increase concrete toughness, usually several times tougher than plain concrete (Wight, 1992). When steel fiber and silica fume are mixed together, they create a synergy effect that results in a stronger chloride enriched impermeable concrete mixture. Superplasticzer is usually added in the mixture to improve FRSF concrete's workability in the low water cement environment.

3.2.1.3 High Performance Concrete Overlay in Alberta

From 1987 to 1989, mixes with 5% silica fume by weight of cement and 60 kg/m^3 steel fibre mixes were placed on small standard bridges. Superplasticizers were not added to the overlay mixture until 1990. Silica fume contents were first increased to 10%, and after that reduced to 8% by weight of cement. According to AIT 2005 specification, the incorporation of silica fume and fly ash in HPC mixture is mandated, while selection of compatible air entraining, water reducing and/or super plasticizing admixtures is discretionary. The general and detailed specifications of HPC are listed in Table 3.2 and Table 3.3, respectively.

Table 3.2 General Specification of High Performance Concrete in Alberta (AIT 2005)

	Requirement
Minimum Specified Compressive Strength at 28 Days (MPa)	45
Size of Coarse Aggregate (mm)	5 - 20
Range of Slump (mm)	90-150
Total Air Content (%)	5-8
Maximum Water/Cement Ratio	0.38

Table 3.3 Detailed Specification of High Performance Concrete in Alberta (AIT 2005)

Item	**Standard Conformed**	**Requirement**
Gradation limit of aggregates	CSA A23.1	Amount of material finer than 160 μm shall be below 5%.
Coarse aggregate	CSA A23.1	
Maximum combination of flat and elongated particles (3:1 ratio) for aggregates	CSA A23.2-13A	Below 10% of the mass of coarse aggregate
Minimum cement content		335 kg/m^3
Total silica fume and fly ash by mass of cementing materials		17 - 20 %
Silica fume by mass of cementing materials		6 - 8 %
Fly ash by mass of cementing materials		11 - 15 %
Slump retention of trial mix after 45 minutes		at least 50% of initial slump
Rapid chloride permeability	ASTM C1202 on laboratory moist cured samples at 28 days	Shall be less than 1000 Coulombs for concrete without steel fibers or concrete containing corrosion inhibiting admixtures.
An air-void spacing factor	ASTM C457	The average of all tests shall not exceed 230 μm with no single test greater than 260 μm

Note: CSA is the abbreviation Canadian Standards Association.

3.2.2 High density concrete overlay (HDOL)

A high density concrete bridge deck overlay (HDOL), has a relatively high content of cement and low content of water. It is also called "Iowa mix" since it was first used by the Iowa Department of Transportation in the 1960s. Typical HDOL has a slump of less than 25 mm and a water/cement ratio of around 0.32, along with the minimum thickness of 44 mm.

The application of HDOL is based on the assumption that the ability of water to penetrate the concrete is related to the water content of the original mix. With less water content, the concrete mixture will have a higher density and be more resistant to water and chloride penetration than conventional concrete. The main disadvantage of HDOL is that its low slump property makes the mixture difficult to place and consolidate, and it is susceptible to weather conditions, extensive hand manipulation and finishing.

HDOL came into use in Alberta in 1978, and AIT has almost stopped using it because of its high shrinkage resulting in extreme cracking problem. As a result, no standard mix design was found in the latest bridge construction specifications. The old specification published by AIT recommended a mix design consisting of 363kg of cement and a 50-50 coarse to fine aggregates. The coarse aggregates were supplied by AIT from their roadway chip coat program. Traditionally, HDOL was mixed in a mobile mixer and placed with a special screed, which could compact zero slump mixture, and where the density was measured as part of the specification with a nuclear densitometer.

3.2.3 Hybrid bridge deck overlay (HYOL)

HDOL bridge overlays, which were constructed between 1955 and 1975 in Alberta, were repaired with thin polymer overlay (TPOL) during 1987 and 1991 by AIT. If HDOL bridges were rehabilitated for the second time in less than 7 years with TPOL, these types of bridge deck overlays are called hybrid bridge deck overlay (HYOL).

There are no specifications published intentionally for construction of HYOL in Alberta. While HYOL is the combination of HDOL and TPOL, the construction of HYOL will be simply following the specifications of HDOL and TPOL respectively.

3.2.4 Asphalt concrete overlay with polymer membrane (ACPM)

Asphalt concrete with polymer membrane (ACPM) is one of the most commonly used protective systems for bridge decks. A bituminous fabric membrane is usually placed on the top of a concrete bridge deck. The edges of the membrane are lapped to resist the water penetration. Between the concrete bridge deck and fabric membrane, a tack coat is applied to enhance the bond strength between the two layers. Several layers of asphalt are then placed on the top of the membrane.

Waterproofing membrane can reduce the oxygen and moisture supply at the rebar level and delay the reinforcement corrosion activity. The combination of asphalt overlay and membrane can prevent water or chloride from penetrating into the original bridge deck. Therefore, it can protect the concrete bridge deck and extend its service life. ACPM can also provide a smooth riding surface and reduce the rate of reflective cracking, even when reflective cracking has already appeared on the top of the bridge deck. However, the service life of ACPM is relatively shorter compared to the high performance concrete overlay.

AIT placed ACPM from 1987 into the middle 1990 by contract. The asphalt concrete membrane overlay in Alberta consists of a 5 mm asphalt membrane, a 5 mm protection board and 2-40 mm lifts of hot-mix asphalt concrete pavement. In regular rehabilitation, only the top 40 mm asphalt concrete overlay is replaced. When overlay is damaged seriously, the whole overlay systems must be removed and completely replaced (AIT, 2003).

The mix design of ACPM depends on each contractor and has changed many times from 1987 to 1992. The mix design of asphalt in ACPM is specified by the 2005 version of standard specifications for highway construction in Alberta. The specifications indicate that the asphalt mix design in Alberta must follow the Marshall Method. The detailed requirement of asphalt content design is listed in Table 3.4.

Table 3.4 Asphalt Concrete Mix Types and Characteristics in Alberta (AIT, 2005)

Mix Type	Aggregate Criteria			Marshall Mix Design Criteria							
	Top Size (mm) (Class for Des. 1 Aggregate)	% MF. -5000 (min) Note 1	% Fractures +5000 (2 faces) (min)	Marshall Stability N (min)	No. of Blows	Flow (mm)	Air Voids (%)	VMA % (min) by % Air Voids		Voids Filled with Asphalt %	Retained Stability % (min)
								3.5	4.0		
H1	16.0	75	98 (one face) 90	12 000	75	2.0 to 3.5	Note 3	13.0	13.5	65-75	70
H2	12.5	70	80	11 500	75	2.0 to 3.5	Note 3	13.5	14.0	65-75	70
M1	12.5	50	60	8 000	75	2.0 to 3.5	Note 3	13.5	14.0	65-75	70
L1	12.5	Note 5	60	5 300	50	2.0 to 4.0	Note 3, 4	13.5	14.0	65-78	70
S1	10.0	Note 5	70	5 300	Note 2	2.0 to 4.0	Note 3	14.5	15.0	65-78	70
S2	10.0	75	90	10 000	75	2.0 to 3.5	Note 3	14.5	15.0	65-78	70
S3	25.0	Note 5	60	8 000	75	2.0 to 4.0	Note 3	11.5	12.0	65-78	70

Design Air Voids	Minimum Theoretical Film Thickness Requirements (µm)	
	Mix Types H1, H2, M1	Mix Type L1, S2, S1 (note 7)
4.0 and 3.9	6.0	6.5
3.7 and 3.8	6.1	6.6
3.5 and 3.6	6.2	6.7
3.3 and 3.4 (L1 for Community Airports only)	-	6.8
3.0, 3.1 and 3.2	-	6.9

Note 1 - The Percentage of Manufactured Fines in the -5000 Portion of the Combined Aggregate.

Note 2 - Use the same number of blows as for the surface course or 50 blows if used as a surface course.

Note 3 - The Design Air Voids shall be chosen as the lowest value, within the range of 3.5 to 4.0% inclusive, such that all other mix design criteria are met.

Note 4 - Air Void limits listed in Note 3 shall be reduced by 0.5% for community airports. VMA at 3.0% Air Voids shall be a minimum of 13.0%. A 300-400A asphalt is normally used for community airports

Note 5 - All fines manufactured by the process of crushing shall be incorporated into the mix.

Note 6- Theoretical Film Thickness shall be as follows, depending on the specified Mix Type and Design Air Voids. The Theoretical Film Thickness value shall be established in accordance with TLT-311.

Note 7 - S1 requirement only for a surface course

3.2.5 Thin polymer overlay (TPOL)

Thin polymer overlay is a thin, flexible, multi-layered, polymer-aggregate wearing surface, which acts as a wearing surface and a membrane system. It consisted of polymer as a binder and a well graded, high quality aggregate as a filler (Jenkins et al., 1977). The typical polymer binders included epoxy, unsaturated polyester styrene and methacrylate. The most widely used aggregates are hard with high quality such as basalt, silica, quartz or granite (Ramcharitar, 2002).

TPOL can reduce the chloride ions and water infiltration and provide an economical skid resistant system for bridge deck. Polymer overlay is preferred when the amount of deck repair is not enormous and the bridge deck is still in fine condition. Polymer overlay is easy to construct with a curing time ranging from 4 to 8 hours. It is typically installed during evening hours and constructed on-site to minimize lane closures and traffic disruption. Compared to other types of overlay, such as HPCO, polymer overlay has a shorter service life.

In 1963, the Bow River Morley bridge deck was the first Alberta bridge to be rehabilitated with polymer overlay. This method was then widely used between 1985 and 1998. The typical structure of polymer overlay is a layer of liquid resin with gaped basalt aggregates on top of it, and with the surface layer of resin tie coat to seal any pinholes. The overlay aggregate should meet the requirement specified by "Specification for Seed Aggregates Used in Polymer Membrane and Overlays (B392)." The physical requirement of polymer is listed in Table 3.5.

Table 3.5 Physical Requirement of Polymer in Alberta (AIT 2005)

PHYSICAL REQUIREMENTS OF POLYMER			
Material	**Physical Property**	**Required Value**	**Test Method**
Polymer	Solids Content	98% (minimum)	ASTM D2369 at 60°C for 2 hours
Polymer	Specific Gravity of Each Component		ASTM D1475
Polymer	Infrared Spectrography and Gas Chromatographic Separation		BT008
Polymer	Bond Strength to Concrete at 7 days	10.0 MPa (minimum)	ASTM C882 Non-sandblasted surface
	Interlayer Bond Strength to Polymer at 2 days. Testes at 23 °C	7.0 MPa (minimum)	
Polymer	Tensile Strength at 7 days	10.0-17.0 MPa	ASTM D638 Speed 4-6 mm/min. Sample type M-1. Use 10×10 mm sample.
	Tensile Strength at 365 days UV exposure	Equivalent of tensile strength at 7days · 3 MPa	
Polymer	Tensile Elongation at 7 days	30% (minimum)	ASTM D638 Speed 4-6 mm/min. Use 10×10 mm sample.
	Tensile Elongation at 365 days UV exposure	20%(minimum)	
Polymer	Modulus of Elasticity at 7 days	900 MPa (maximum)	ASTM C109 (Modified) 50×50 mm sample
Polymer Mortar	Compressive Strength at 7 days	40.0 MPa (minimum)	ASTM C109 (Modified) 50×50 mm sample
Polymer Mortar	Thermal Compatibility at 7 days	10 cycles of -21 °C to 60 °C (minimum)	ASTM C884 6mm depth
Polymer Mortar	Absorption Volume of Permeable Voids at 7 days	1.25% (maximum)	ASTM C642 50×50 mm cubes oven dry at 60 °C for 48 hours

Table 3.6 Minimum Polymer Coverage Requirements in Alberta (AIT 2005)

Minimum Polymer Coverage Requirements (l/m^2)			
Wearing Surface Class	**1st Layer**	**2nd Layer**	**3rd Layer**
A	1.33	2.00	0.30
B	1.33	2.00	N/A
C	1.33	0.30	N/A

3.3 Bridge Deck Service Life Prediction Models in Alberta

AIT is one of the leading highway agencies that specializes in developing bridge deck service life prediction models. In the early 1980's, AIT developed a service life prediction model for un-repaired bridge decks with CSE test results. This model was named the AT model. As time went on, more and more bridge decks were repaired. A new service life prediction model was then developed for repaired bridge decks by Skeet and Kriviak in 1994. This model was herein called ET model. The ET model was adjusted several times based on new CSE test results.

The AT model assumes that CSE values after rehabilitation will increase regardless of the decrease effect caused by rehabilitation. This model is mainly applied when predicting the service life of non-repaired bridge decks. The ET model conducts regression analysis based on the historical CSE test data and considered impact factors such as traffic, freeze-thaw cycle, etc. Therefore, the ET model is more accurate in predicting service life of rehabilitated bridge decks, compared to the AT model.

Figure 3.1 shows one example of AT and ET model service life prediction curves. This bridge deck was overlaid by silica fume concrete in 1995. The AT model predicts CSE value over time after year 2001, when the latest CSE test was conducted after bridge deck rehabilitation. AT model is represented by a straight line. ET model predicts the relationship between CSE values after rehabilitation over time and is represented by a solid curve. In order to improve the prediction accuracy, this solid curve is adjusted to dashed curve by shifting the curve to the latest CSE test data.

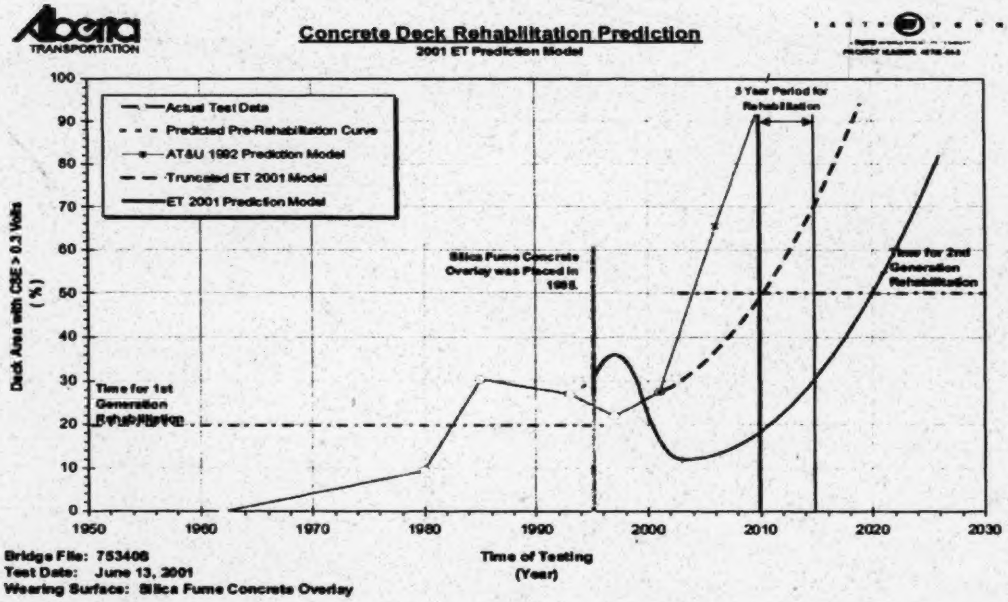

Figure 3.1 Service Life Prediction Curves Generated by AT and ET Models

The detailed AT and ET models description will be introduced in the following sections.

3.3.1 AT model

In the 1980's, the Alberta Transportation (AT) model was developed to predict service life of bridge deck overlays in Alberta, during which there were few bridge decks repaired. The model mainly focused on predicting the service life of non-repaired bridge decks. The model assumed that CSE value after rehabilitation will increase regardless of the decrease effect caused by rehabilitation.

3.3.2 ET model

In 1994, Skeet and Kriviak (1994) conducted a study on protective systems service life prediction for concrete bridge decks for AIT. This model was later revised by

Earth Tech Company in 2001 and was called the ET model. Author and report name of these two reports are listed in table 3.7.

Table 3.7 Reports of ET Models

Year	Author	Report Name	Report Number
1994	Skeet and Kriviak from Reid Crowther and Partners Ltd	Service Life Prediction of Protective Systems for Concrete Bridge Decks in Alberta	ABTR/RD/R R-94/01
2001	Earth Tech (Canada) Inc	Deck Rehab Database Draft Report	4371801

1994 Study

This study began by defining the following two concepts: repairable and failure service life. Repairable service life was a period of time during which deterioration could be identified and measured damage quantities were so low that repairs could be taken (Skeet and Kriviak, 1994). Damage quantities were defined as debonded or delaminated bridge deck or overlay areas detected by chain drag test or bridge deck area that had been moved during repair. AIT considered repairable service life of bridge deck/overlay was reached when the total damage achieved 5%.

Failure service life was defined as "a period of time during which deterioration is too great to optimally undertake repairs and instead deterioration is allowed to continue until unsafe or unacceptable user conditions are reached and replacement of the system or structure was required" (Skeet and Kriviak, 1994). Reinforcement rebars corrosion and overlay debonding were identified as the main bridge deck/overlay failures mechanisms, both resulting in bridge deck spalling. Failure service life of bridge deck/overlay was reached when spalling achieved 5%.

The following four models were developed by Skeet and Kriviak based on the available data at that time:

1. Relationship between the percent of deck area damaged and percentage of total deck area with CSE more negative than specific level. This made it possible to predict service life with CSE data.
2. Relationship between average bridge deck CSE value and percent of deck with CSE < –300 mV.
3. Model of corrosion overtime, i.e. relationship between percent of deck with CSE < –300 mV and time and relationship between average CSE and time.
4. Conceptual model of overlay debonding over time, i.e. relationship between percentages of deck area debonded and time (Skeet and Kriviak, 1994).

Of all the factors which have an impact on the service life of bridge deck/overlay, the following four factors were took into consideration in corrosion and debonding service life prediction:

1. Structure type
2. Average daily traffic
3. Environmental Condition, freeze-thaw cycle, and
4. Geometric design, mainly slope (drainage)

Table 3.8 and 3.9 show concrete bridge deck overlay corrosion and debonded service life prediction results. Expert opinions were used to consider the impact of the above four factors on service lives of bridge decks in Alberta.

Table 3.8 Corrosion Service Life Prediction (Skeet and Kriviak, 1994)

Protection System	Basic Average Service Life of Protection Systems From Start of Corrosion		Adjustments to Average Service Life for Various Factors										
			Structure Type			Loading Conditions (ADT)			Environmental Conditions (Freeze/Thaw Cycles)		Drainage Conditions (crossfall/gradeline)		Epoxy W.S.
	Optimum Repair of 5% Damage	To Failure of 5% Spalling	Concrete Girders Cip Deck	Prestress Girders P/C Deck	Steel Girders Cip Deck	Primary HWY >5000	Secondary HWY	Local Road <600	High >115	Low <90	Bad xfall <1% Grade <1%	Good Xfall >2% Grade >2%	
AS CONSTRUCTED													
Exposed Concrete Decks - Pre1975	20	30	+1	+6	-3	-5	0	+10	-3	+2	-2	+2	+16
Asphalt Covered Decks	18	32	0	+4	-3	-7	-1	+7	-4	+2	-3	+2	n/a
Never Repaired LSDC Overlay	20	40	0	+2	-5	-6	0	+12	-3	+3	-3	+3	+10
AS REPAIRED													
Silica Fume Concrete Overlay	*18*	*35*	*0*	*+4*	*-5*	*-5*	*0*	*+10*	*-3*	*+3*	*-3*	*+3*	*+15*
Thin Latex Modified Concrete Overlay	*10*	*20*	*0*	*+2*	*-6*	*-7*	*0*	*+8*	*-3*	*+4*	*-3*	*+3*	*+15*
Rapid Set Concrete Overlay	*16*	*25*	*0*	*+4*	*-4*	*-6*	*0*	*+10*	*-3*	*+5*	*-3*	*+3*	*+15*
Silica Fume/Fibre Reinforced Concrete Overlay	*20*	*40*	*+2*	*+6*	*-2*	*-4*	*0*	*+12*	*-3*	*+6*	*-2*	*+3*	*+15*
Membrane and ACP Overlay	*16*	*28*	*0*	*+4*	*-3*	*-5*	*0*	*+7*	*-3*	*+7*	*-4*	*+3*	*n/a*
50 mm LSDC Overlay	*15*	*27*	*0*	*+4*	*-5*	*-6*	*0*	*+10*	*-3*	*+8*	*-3*	*+3*	*+15*

Notes: Italics indicates derived by expert opinion

Table 3.9 Concrete Overlay Debonding Service Life Prediction (Skeet and Kriviak, 1994)

Protection System	Basic Average Service Life of Protection Systems From Start of Corrosion		Adjustments to Average Service Life for Various Factors											
			Structure Type				Loading Conditions (ADT)			Environmental Conditions (Freeze / Thaw Cycles)		Drainage Conditions (crossfall/gradeline)		Epoxy W.S.
	Optimum Repair of 5% Damage	To Failure of 5% Spalling	Flexible cip Decks	Less Flexible cip Decks	Connect Precast Girders	Unconnect Precast Girders	Primary HWY >5000	Secondary HWY	Local Road <600	High >115	Low <90	Bad xfall < 1% Grade <1%	Good xfall>2 % Grade >2%	
50 mm LSDC Overlay	*25*	*40*	*0*	*+3*	*+7*	*-7*	*-5*	*0*	*+5*	*-3*	*+3*	*-3*	*+3*	*+12*
Silica Fume Concrete Overlay	*32*	*42*	*0*	*+3*	*+7*	*-7*	*-5*	*0*	*+5*	*-3*	*+3*	*-3*	*+3*	*+12*
Thin Latex Modified Concrete Overlay	*25*	*40*	*0*	*+3*	*+7*	*-7*	*-5*	*0*	*+5*	*-3*	*+3*	*-3*	*+3*	*+12*
Rapid Set Concrete Overlay	*25*	*40*	*0*	*+3*	*+7*	*-7*	*-5*	*0*	*+5*	*-3*	*+3*	*-3*	*+3*	*+12*
Silica Fume/Fibre Reinforced Concrete Overlay	*32*	*52*	*0*	*+3*	*+7*	*-7*	*-5*	*0*	*+5*	*-3*	*+3*	*-3*	*+3*	*+12*

Notes: Italics indicates derived by expert opinion

2001 Study

There were only 1,580 independent data inspection records from 547 bridge sites in Concrete Deck Information System (CDIS) in 1994. Seven years later, CDIS was expanded to 2,604 separate test inspection records from 591 different bridge sites. A new study, herein 2001 study, was conducted by Earth Tech (Canada) to take into account of these new inspection records. The ET model was also adjusted. Two other factors, as stated below, were considered besides the four factors mentioned in the 1994 study:

1. Timing of repair (CSE level at time of repair)
2. Quality of repair (based very simply on crack frequency).

Two performance indicators, rehabilitation life and life gained, were used in the 2001 study to evaluate the effectiveness of deck repair methods. The definition of these two performance indicators are explained below.

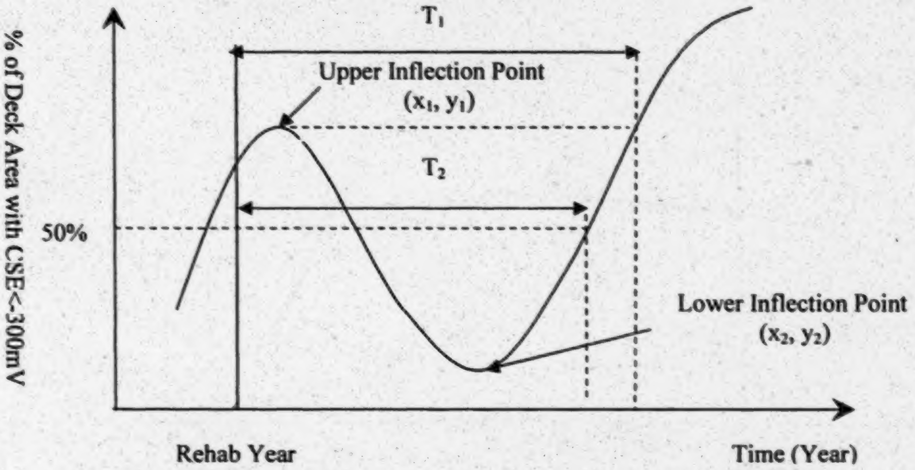

Figure 3.2 Typical CSE Data Development Curve after Rehabilitation

As shown in Figure 3.2, the following five important concepts were defined as (Earth Tech, 2001):

1. Upper Inflection point (x_1, y_1): the highest point on the CSE curve just following rehabilitation, where x_1 indicates rehab year, and y_1 indicates percentage of deck area with CSE less than -300mV;

2. Lower inflection point (x_2, y_2): the lowest point on the CSE curve after rehabilitation, usually 5 to 10 years after rehab;
3. First half of post-rehab time T, which is (x_2-x_1);
4. Rehab life (T1): The time (years) from the time of rehab to the point on the CSE curve where CSE returns to the y_1 level;
5. Life gained by rehab (T2): The time (years) from the time of rehab to the point on the curve where CSE returns to the 50% of deck area with CSE < -300 mV level. This level of corrosion in deck is assumed to correspond to 5% spalling and another repair time for bridge deck.

Two measures of performance that were used in 2001 were:

1. Increase in service life resulting from the rehabilitation,
2. Drop in post-rehab CSE readings.

As shown in Table 3.10, the 2001 study indicated that HYOL bridges had the longest first half of post-rehab time T, time for CSE to return to initial level T1 and time for CSE to return to 50% level T2. Four other types of bridge deck overlays had very close values of T, T1 and T2. Therefore, it was not applicable to simply rank the effectiveness of these bridge deck overlay types by these three values.

Table 3.10 Effectiveness Comparisons of Five Different Bridge Deck Overlays in 2001 Study

	HPCO	HDOL	HYOL	ACPM	TPOL
Time to Lower Inflection Point T (years)	6.91	7.37	12.78	7.42	6.28
Time for CSE to return to initial level T1 (Years)	15.13	16.51	27.48	15.06	15.83
T1/T Ratio	2.19	2.24	2.15	2.03	2.52
Time for CSE to return to 50% level T2 (years)	22.87	16.66	19.94	20.11	19.28
T2/T Ratio	3.31	2.26	1.56	2.71	3.07

CHAPTER 4 EFFECTIVENSS OF BRIDGE DECK REHABILITATION METHODS IN ALBERTA

This chapter compares the effectiveness of five different concrete bridge deck rehabilitation overlay methods used in Alberta. The main bridge deck overlay effectiveness indicator is corrosion service life, which is predicted by Alberta Transportation (AT) and Earth Tech (ET) models. Statistical and expert opinion methods are used in effectiveness comparisons. The impact of traffic and rehabilitation time on the bridge deck service life is also discussed.

This is an important study as it gives Alberta Infrastructure and Transportation (AIT) a better understanding of the effectiveness of its bridge deck rehabilitation methods. In addition, these findings could be used in network level bridge deck rehabilitation management and planning.

4.1 Effectiveness Analysis by Service Life Prediction Models

Life cycle cost analysis (LCCA) is used in the selection of infrastructure and building rehabilitation alternatives as it considers overall cost of rehabilitation alternatives during their service life. However, due to the lack of life cycle cost data, service life would be a primary indicator in comparing the effectiveness of bridge deck rehabilitation overlay methods in this study.

Rehab life T1 was defined as the duration between time of rehabilitation and percentage of deck area with CSE < -300 mV after rehabilitation returns to the level of rehabilitation. If the rehabilitation is very effective, then the percentage of CSE level will drop significantly and it will take a very long time for the bridge deck overlay to return back to the original level. The longer the T1 value is, the more effective the bridge deck overlay will be. Therefore, T1 is chosen as one indicator for bridge deck overlays effectiveness comparison.

Skeet and Kriviak (1994) defined repairable service life and failure service life in their study, as explained in Chapter 3. No model was developed to predict these two service lives directly. However, it was assumed that a repairable service life was reached when 50% of the deck area was with CSE < -300 mV. For overlaid bridges, the duration between the time of this repair and the time of 50% of deck area was with CSE < -300 mV and is represented by life gained by rehab (T2) value, which can be calculated from the service life prediction curve generated by AT and ET models. T2 value reflects the repairable service life of bridge deck overlays and, therefore, can be used for bridge deck overlays effectiveness comparison. The longer the repairable service life, the more effective the bridge deck rehabilitation overlay methods will be.

Section 4.1.1 will validate AT and ET models by comparing a predicted percentage of deck area with CSE < -300 mV value with a measured value. Section 4.1.2 will introduce bridge deck repair performance indicators (T1 and T2) database for all studied bridges, followed by the bridge deck overlays effectiveness comparison.

4.1.1 Models validation

In 1998, Earth Tech generated bridge deck service life prediction curves for some bridges based on AT and ET models, forecasting a percentage of deck area with CSE < -300 mV over a given time. From 2001 to 2002, CSE tests were conducted on some bridges and test results included average CSE test value and percentage of deck area with CSE < -300 mV. It was decided to investigate the accuracy of AT and ET service life prediction models against measured values in 2001 and 2002. In order to do this, percentage of deck area with CSE < -300 mV from CSE testing during 2001 and 2002 were compared with predicted values for the same parameter from ET and AT models.

One hundred and thirty-one bridges, with both predicted and measured value available, were selected for validating service life models. Sixty-three of those bridges were tested in 2001 and 68 bridges were tested in 2002. A list of these bridges, along with predicted and measured CSE test results, and test year information are reported. Results of this comparison study have been reported in Appendix A. Table 4.1 shows a part of the results from Appendix A.

Table 4.1 Predicted and Measured Values of Percentage of Deck Area with CSE less than -300 mV for Some Bridges

Bridge File	**Percentage of Deck Area with CSE Less Than -300mV**		
	Measured Value	**Predicted Value by ET Model**	**Predicted Value by AT Model**
167	34.3	30	56
233	17.9	11	20
272	3.8	10	36
2233	2	12.5	13
2401	17.2	21	56
6809	0.6	0	48
7295	0	60	50
7461	10	12	50
....	...	...	...

It is important to discover the criteria for good service life prediction for these models. The mean value comparison of measured and predicted values for groups of bridges can not present the accuracy of the model prediction. Hence, differences between ET or AT predicted and measured values were calculated. In this study, a good prediction was defined when the absolute difference between predicted value and measured value was less than 20 percent for bridge deck area with CSE < -300 mV parameter.

Frequency and cumulative distributions of difference between AT predicted value and measured value are listed in Table 4.1. AT model performed well, since 91% of the bridges had an absolute difference between predicted value and measured value of less than 20 percent of bridge deck area with CSE < -300 mV. Hence, it is concluded that the AT model is reliable in predicting the percentage of deck area with CSE < -300 mV value.

Table 4.2 Frequency and Cumulative Distribution of Absolute Difference of AT Predicted and Measured CSE Values

Bin	Frequency	Cumulative Distribution (%)
-100	0	0.00
-80	1	0.76
-60	3	3.05
-40	5	6.87
-20	19	21.37
0	85	86.26
20	15	97.71
40	3	100
60	0	100
80	0	100
100	0	100

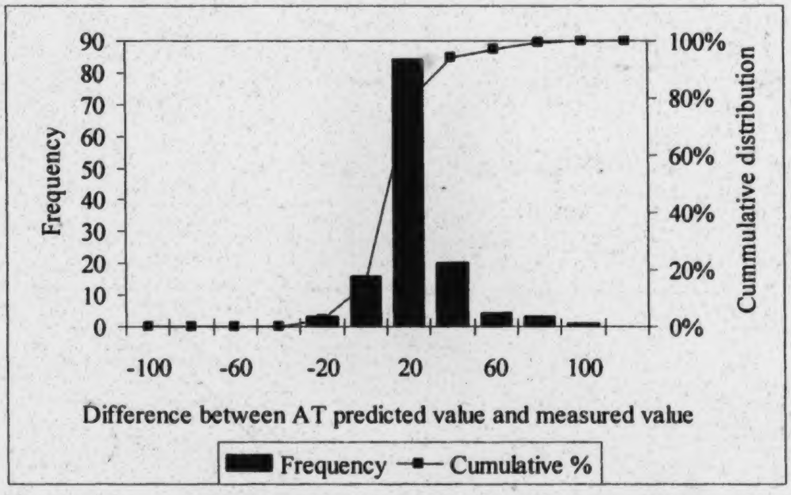

Figure 4.1 Histogram of Difference between AT Predicted Value and Measured Value on Percentage of deck area with CSE < -300 mV

The same process was undertaken to validate the ET model. Absolute differences between ET models predicted values, and measured values, were calculated. Frequency and cumulative distributions of the absolute difference are listed in Table 4.3. The ET model performaned well, since 74% of the bridges had an absolute difference between predicted value and measured value of less than 20 percent of bridge deck area with CSE < -300 mV. Hence, it is concluded that the ET model is also reliable in predicting the percentage of deck area with CSE < -300 mV value.

Table 4.3 Frequency and Cumulative Distribution of Absolute Difference between ET Predicted and Measured CSE Test Values

Bin	**Frequency**	**Cumulative (%)**
-100	0	.00
-80	0	.00
-60	3	2.29
-40	4	5.34
-20	12	14.50
0	47	50.38
20	38	79.39
40	14	90.08
60	10	97.71
80	2	99.24
100	1	100

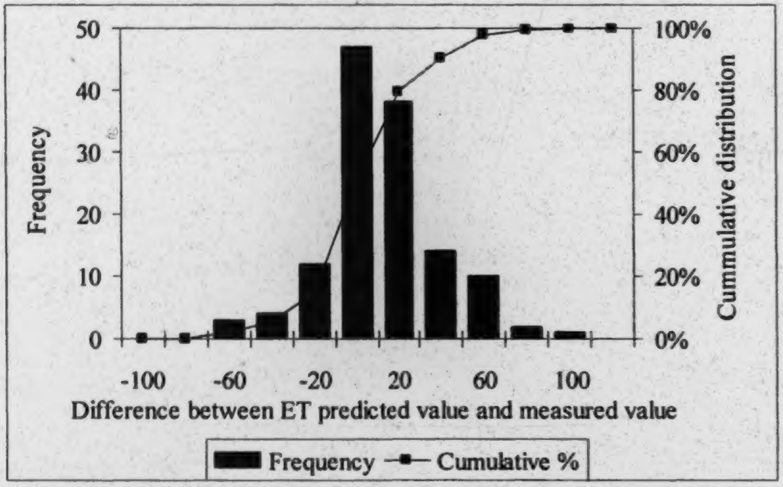

Figure 4.2 Histogram of Difference between ET Predicted Value and Measured Value on Percentage of deck area with CSE < -300 mV

4.1.2 Data collection

AIT conducted CSE field tests on 313 bridges from 2001 to 2003. The number of bridges tested in each year is listed in Table 4.4.

Table 4.4 Number of Bridges Tested in Each Year

Year	Number of Bridges
2001	85
2002	95
2003	43
2004	90
Total	313

Bridges were eliminated from this study if they had no rehabilitation in their history or were not rehabilitated by any of the studied overlay methods, as listed in Table 4.5. Of all the 313 bridges, 71 bridges were eliminated and the rest, 242 bridges, were then categorized into five groups by overlay types. The number of bridges in each group is listed in Table 4.5.

Table 4.5 Number of Bridges in Each Group

Bridge Deck Overlay Types	Number
High Performance Concrete Overlay (HPCO)	95
High Density (Low Slump) Concrete Overlay (HDCO)	42
Hybrid Overlay (HYOL)	23
Asphalt Concrete with Polymer Membrane (ACPM)	35
Thin Polymer Overlay (TPOL)	47
Total	242

Two service life indicators, rehab life (T1) and life gained by rehab (T2), were extracted from the service life prediction curve generated by AT and ET models. However, T1 or T2 data were not available for all bridges. For instance, Bridge 74233 has none of these two data available. One reason was that the rehabilitation was so effective that the percentage of deck area with CSE < -300 mV kept dropping after rehabilitation. Therefore, no lower inflection point occurred and the percentage of deck area with CSE < -300 mV would never return to the level of upper inflection point. As a result, it was impossible to calculate the T1 value. Furthermore, the percentage of deck area with CSE < -300 mV always stayed in a low level and never reached to 50 percentage, thus T2 could not be calculated.

Bridges with T1 and T2 data unavailable were eliminated from this study. In total, around 140 bridges were investigated, which had T1 and T2 data available from ET model, and around 175 bridges from the AT model. Table 4.6 lists the number of bridges with T1 and T2 data available for each overlay type.

Table 4.6 Number of Bridges with T1 and T2 Data Available For Each Overlay Type

Rehab Type	**Service life predicted by ET Model**		**Service life predicted by AT Model**	
	T1 (years)	**T2(years)**	**T1 (years)**	**T2 (years)**
HPCO	59	58	70	70
HDOL	24	24	31	31
HYOL	15	15	15	15
ACPM	21	21	29	28
TPOL	24	24	32	32
Total	143	142	177	176

CSE testing is not conducted every year after rehabilitation. Thus, it is highly possible that no test was conducted around the time of the upper inflection point. Interpolation and judgement are the two main methods that estimate the upper inflection point. T1, therefore, is to some extent subjective because it is determined by the position of upper inflection point. T2 is determined by 50% of deck area with CSE < -300mV and is more deterministic, compared to T1. As part of Appendix B, Tables 4.7 presents T1 and T2 values calculated from the AT and ET models for HUOL overlaid bridges. The first column represents the bridge file number, which is coded by AIT. The following six columns represent times of repair as well as repair type and repair year. From left to right, the last four columns represent T1 by ET model, T1 by AT model, T2 by ET model and T2 by AT model, respectively.

Table 4.7 Bridges Rehabilitated with Hybrid Overlay (HDOL and Polymer Overlay)

Bridge File Number	1st Repair		2nd Repair		3rd Repair		Rehab life (T1)		Life gained by rehab (T2)	
	Type	Year	Type	Year	Type	Year	ET	AT	ET	AT
233	HDOL	1982	Polymer OL	1989			34	28	29	24
2233	HDOL	1981	Epoxy OL	1986	Deck Sealed+Chip Coat	1992+2001	50	30	54	30
8495	HDOL	1986	Deck Sealed	1988	Poly OL	1992	16	16	6	6
8719	HDOL	1985	Epoxy OL	1991	Chip Coat	1997	25	22	18	17
74233	HDOL	1981	Epoxy OL	1988	Deck Sealed+Chip Coat	1990+1997	-	-	-	-
74352W	HDOL	1984	Polymer OL	1990	Chip Coat	1995	25	25	22	22
74353W	HDOL	1983	Epoxy OL	1987	Chip Coat	1995	24	29	23	25
75070	HDOL	1982	Polymer OL	1987	Chip Coat	1993	42	29	44	30
70935	HDOL	1984	Polymer OL	1990	Chip Coat	1999	23	22	20	19
73407	HDOL	1985	Polymer OL	1989	Chip Coat	1998	26	16	6	6
74353E	HDOL	1985	Epoxy OL	1989	Chip Coat	1995	27	26	28	27
74354W	HDOL	1982	Polymer OL	1988	Chip Coat	1995	32	28	30	27
75186	HDOL	1986	Polymer OL	1990	Chip Coat	1998	16	16	25	19
75932	HDOL	1984	Epoxy OL	1990	Chip Coat	1994	21	23	18	18
75555	HDOL	1985	Epoxy OL	1989	Chip Coat	1993	35	23	25	19
75677	HDOL	1982	Polymer OL	1988	Chip Coat	1994	58	25	33	21

4.1.3 Comparison between rehabilitation methods by models

Figure 4.3 shows the effectiveness of different bridge deck overlays in terms of average rehab life of T1 and life gained by using rehab T2. The longer the T1 and T2, the more effective the bridge deck overlay is.

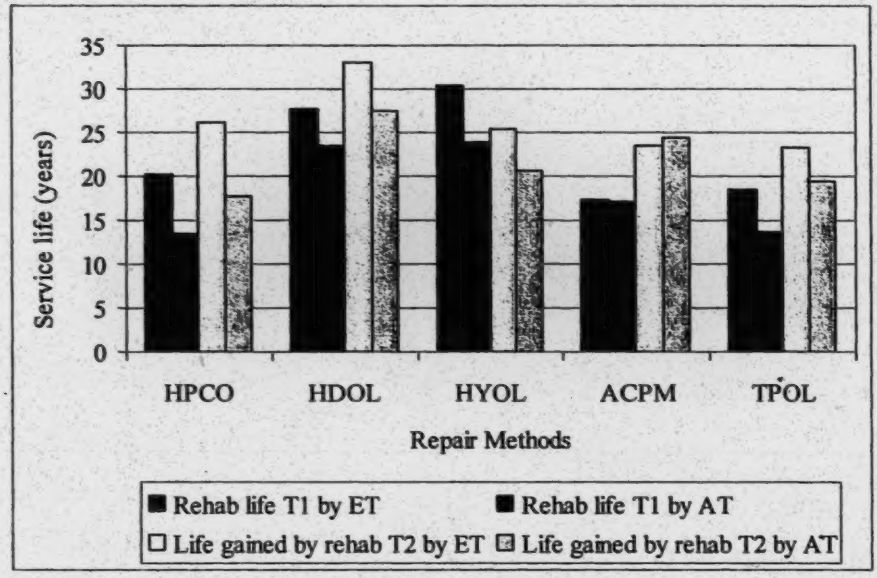

Figure 4.3 Averages of Effectiveness Indicators (T1 and T2) for Five Different Bridge Deck Repair Methods in Alberta

Statistical analysis results for an effective comparison, based on T1 and T2, for five rehabilitation methods including mean, standard deviation, minimum maximum of T1 and T2, and number of observations of each bridge deck overlay type as summarized in Table 4.8.

Table 4.8 Average Repair Effectiveness Indicators (T1 and T2) For Five Different Bridge Deck Repair Methods in Alberta

Repair Method	**Statistical Parameters**	**Rehab life T1 (years)**		**Life gained by rehab T2 (years)**	
		ET	**AT**	**ET**	**AT**
HPCO	Mean	20.2	13.5	26.1	17.6
	Standard Deviation	8.0	4.3	10.1	4.4
	Min.	7	5	6	6
	Max.	47	23	49	24
	Number of Observations	59	58	70	70
HDOL	Mean	27.7	23.5	33.1	27.5
	Standard Deviation	7.5	5.6	10.5	7.7
	Min.	13	13	14	14
	Max.	46	36	50	49
	Number of Observations	24	24	31	31
HYOL	Mean	30.3	23.0	25.4	20.0
	Standard Deviation	12	4.8	12.5	7.2
	Min.	16	16	6	6
	Max.	58	30	54	30
	Number of Observations	15	15	15	15
ACPM	Mean	17.4	17.0	23.5	24.4
	Standard Deviation	5.2	6.9	8.0	8.0
	Min.	6	6	12	11
	Max.	27	34	53	40
	Number of Observations	21	21	29	28
TPOL	Mean	18.5	13.6	23.3	19.4
	Standard Deviation	10.1	6.0	9.5	7.2
	Min.	8	3	9	8
	Max.	44	31	45	38
	Number of Observations	24	24	32	32

Several conclusions can be drawn based on Figure 4.3 and Table 4.8.

1. For all these five types of bridge deck overlays, AT and ET models give a close prediction value on rehab life (T1) and life gained by rehab (T2). In terms of effectiveness of bridge deck overlay, HDOL is the most effective and TPOL is the least effective. The mean value of T2, predicted by the ET model, is 33.1 years for HDOL and 19.4 years for TPOL.

2. For these five bridge deck overlays, the standard deviation of T2, predicted by the ET model, is in the range of 8.0 to 12.5 and the mean value is in the range of 23.3 to 33.1. This indicates that a high deviation exists in T2 data of all these five groups.

3. AT and ET models have different prediction results for T1 and T2. For example, for HPCO, the mean value of T2 is 26.1 by ET model, and 17.6 by AT model. The difference between these two models is 8.5 years, which is high compared to the mean value. But for ACPM, these two models have a very close prediction with the difference of only 0.9 years.

4. Rehab life T1 is lower than life gained by rehab T2 for all groups of bridge deck overlays except HYOL.

4.2 Comparison by Expert Opinion Methods

In section 4.1, the AT and ET models were used to estimate the effectiveness of bridge deck overlay methods by rehab life T1 and life gained by rehab T2. Another method for bridge deck rehabilitation service life estimation could be expert opinion. Expert opinion is a good supplementary method to assess the bridge deck effectiveness because experts have the ability to consider all the factors that have an impact on the effectiveness of bridge deck overlays. The limitation of this method is that the survey result could vary by type of questions and selection of experts in the study.

A questionnaire was developed to find out bridge experts' opinion on bridge deck overlay service life. It is widely accepted that traffic has a great impact on the service life of a bridge deck, i.e. higher traffic results in shorter service life. Therefore, in questionnaire design, traffic conditions were divided into the following three levels: high traffic level (average daily traffic more than 5000); intermediate level (average daily traffic more than 1000 and less than 5000); and low traffic level (average daily traffic less than 1000). Experts were asked to estimate the service life of four different bridge deck overlays including the following: HPCO, HDOL, ACPM and TPOL. A copy of the questionnaire is shown in Appendix B. The questionnaire was sent to more than 10 bridge experts selected from Alberta Infrastructure and Transportation regional and central offices and bridge engineers from consulting companies. Table 4.9 shows the number of experts who responded to the questionnaire.

Table 4.9 Respondents Number for Bridge Deck Overlay Service Life Estimation

Type	Worst Case (ADT > 5000)	Typical Case (ADT 1000-5000)	Best Case (ADT<1000)
HPCO	10	10	10
HDOL	10	10	10
ACPM	9	8	8
TPOL	10	10	10

Table 4.10 summarizies the average and standard deviation for service life of four bridge deck rehabilitation methods in Alberta, based on responses from experts.

Table 4.10 the Summarization of Different Bridge Deck Overlays Service Life by Expert Opinions

Method of Repair		**Service Life**			
		Worst Case (AADT > 5000)	**Typical Case (AADT 1000-5000)**	**Best Case (AADT<1000)**	**Total Average**
HPCO	Average	24.2	29.6	33.7	29.1
	Standard Deviation	0.18	0.16	0.22	
HDOL	Average	20.7	25.75	30.2	25.5
	Standard Deviation	0.18	0.18	0.22	
ACPM	Average	12.2	16.8	20.7	16.6
	Standard Deviation	0.20	0.17	0.26	
TPOL	Average	13.2	16.0	19.4	16.2
	Standard Deviation	0.62	0.56	0.49	

Figures 4.4 to 4.6 are box plots of estimated bridge deck service life estimation results by expert opinion. Box plot is a graphical tool for examining one or more sets of data. This graph mainly summarizes the following five statistical measures:

- Median: the line in the middle of the box.
- Upper and Lower quartiles: the upper edge of the box indicates 75th percentile, the lower edge of the box indicates 25th percentile.
- Maximum and minimum values: the bar above the box indicates maximum value and the bar below the box indicates the minimum value, only if

outliers occur in which case the bar extend to a maximum of 1.5 times box length. The box length is the interquartile range.

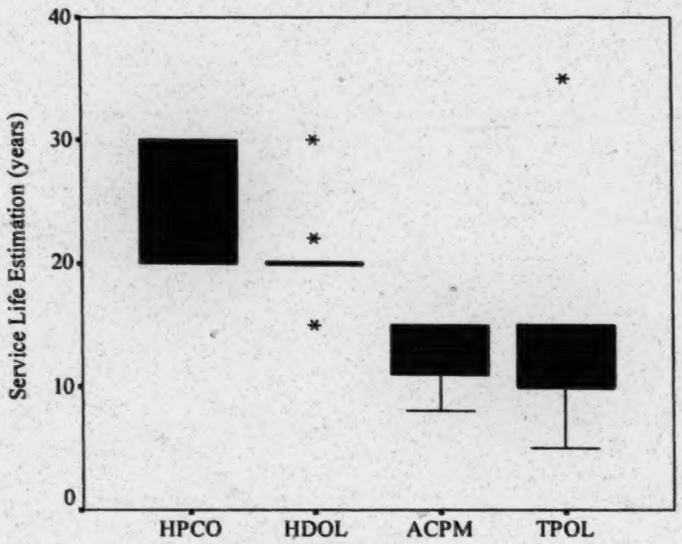

Figure 4.4 Box Plot of Bridge Deck Overlay Service Life for Bridges with ADT > 5000

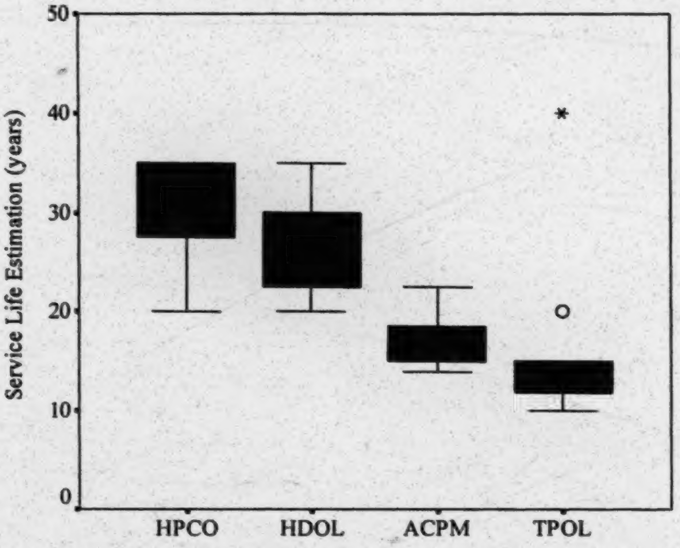

Figure 4.5 Box Plot of Bridge Deck Overlay Service Life for Bridges with ADT between 1000 and 5000

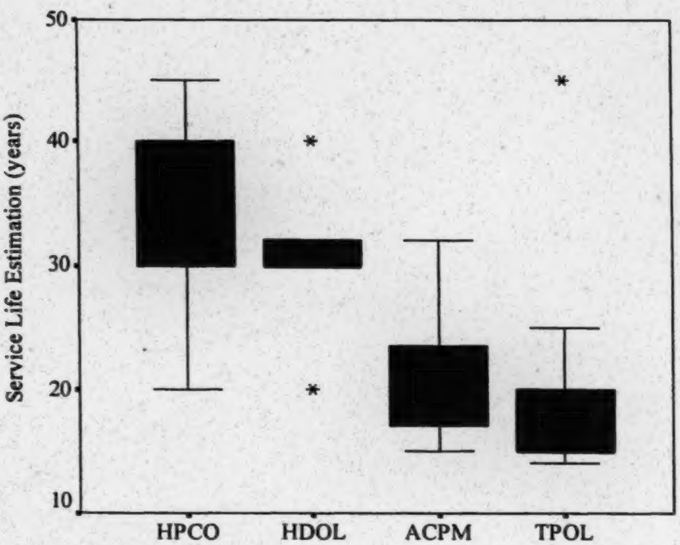

Figure 4.6 Box Plot of Bridge Deck Overlay Service Life for Bridges with ADT< 1000

The following several conclusions could be drawn from Table 4.10 and the box plot figures:

1. HPCO has the highest service life. The sequence of bridge deck overlay service life by median values is high performance concrete overlay, high density concrete overlay, asphalt concrete with polymer membrane and thin polymer overlay.

2. Asphalt concrete with polymer membrane and thin polymer overlay has very close median service life value, which means that they have very close effectiveness.

3. By increasing traffic volume from one level to another level, the service life of all bridge deck overlays will decrease about 5 years.

4. There is one outliner in each box plot graph for TPOL, which comes from one expert who works in southern Alberta. These experts gave a much

higher service life prediction than other experts. This could be due to dry and sunny weather in southern Alberta where less de-icing salt is applied.

4.3 Two Methods Results Comparison

The performance prediction models and expert opinion gave different estimations on the service life of bridge deck overlays as shown in Table 4.11.

Table 4.11 Compare the Estimated Service Life by Performance Models and Expert Opinion

Repair Methods	**Expert Opinion (years)**	**Life Gained by Rehab T2 (years)**	
		ET Model	**AT Model**
HPCO	29.1	26.1	17.6
HDOL	25.5	33.1	27.6
ACPM	16.2	23.5	24.4
TPOL	16.6	23.3	19.4

The following are several conclusions that could be drawn:

1. Service life predicted by the ET model is closer to that by the expert opinion, comparing to AT model;
2. Both performance models and expert opinion agree that ACPM and TPOL are least effective in terms of service life;
3. ET model gives a very close service life prediction to expert opinion for HPCO, with 3 years of service life difference;
4. For HDOL, service life predicted by performance models is higher than that by expert opinion. As mentioned in Section 3.2, bridges repaired by high density concrete overlay are categorized into two groups, HDOL and HYOL. Bridges in HYOL group are repaired twice in less than seven years due to the serious split problem. Therefore, they have relatively bad effectiveness and less service life. Bridges in HDOL group have better performance and have longer service life. This is the reason that service life HDOL, predicted by the AT and ET model, has a higher service life than that by expert opinion.

4.4 Impact of Factors

By reviewing the literature, it was found that there are several factors which have impact on the corrosion of reinforcement in concrete bridge decks. Detwiler (1997) conducted an experiment to investigate the impact of four factors on the quality of bridge deck overlays. These four factors were aggregate type, surface preparation, bonding slurry and substrate age. Madanat (1997) identified several factors that had an impact on bridge deck deterioration such as bridge type, span length, protective system type, skewness, environmental factor, traffic volume and age. Chase et al. (2000) tried to develop a model to find out the impact of several factors on condition rating. These factors were bridge age, average daily traffic, environmental factors and construction material type. Environmental factors included precipitin, temperature range, number of freeze and thaw cycles, and frequency of salt application. In this study, the impact of two specific factors, traffic and bridge age at the time of rehabilitation, were investigated regarding the effectiveness of bridge deck overlay. These two factors were selected based on availability of data for tested bridges in Alberta.

It is generally recognized in infrastructure management system that the rehabilitation time has a great impact on the effectiveness of rehabilitation methods. If the rehabilitation is taken at an early age of deterioration, infrastructure condition will be improved significantly at relatively low costs, and the service life of the bridge deck will be extended greatly. However, if the rehabilitation is taken when the infrastructure is in very poor condition, the rehabilitation costs will increase significantly and the effectiveness of the rehabilitation methods will be very low. The same impact is expected for the rehabilitation time on the effectiveness of bridge deck overlays.

Sections 4.4.1 and 4.4.2 will attempt to analysis the impact of rehabilitation time on effectiveness indicators, the rehab life T1 and life gained by rehab T2. Sections 4.4.3 and 4.4.4 will try to investigate the impact of average daily traffic on the effectiveness indicators, rehab life (T1) and life gained by rehab (T2).

4.4.1 Impact of repair time on rehab life T1

It is expected that rehab life T1 and percentage of deck area with CSE < -300mV at rehab time has inverse proportional relationship. However, the linear trendline of Figure 4.7 shows a poor relationship between the percentage of deck area with CSE < -300mV at rehab time and T1 for all the bridges studied in this research. The same relationships for each repair method, HPCO, HDOL, ACPM and TPOL, are shown in Figure 4.8, 4.9, 4.11 and 4.12. Only Figure 4.10 shows, to some extent, an indirect proportional relationship for group HYOL. As mentioned before, the selection of upper inflation point is subjective, which may result in unexpected relationship trends.

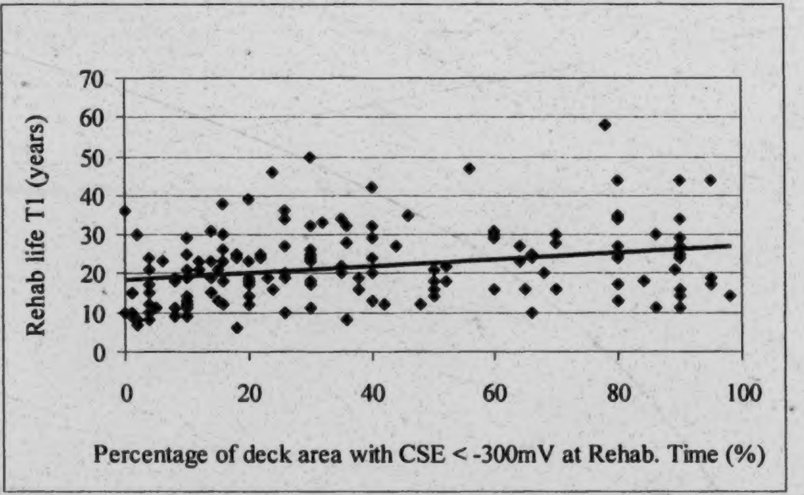

Figure 4.7 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Rehab Life T1 for All Bridges

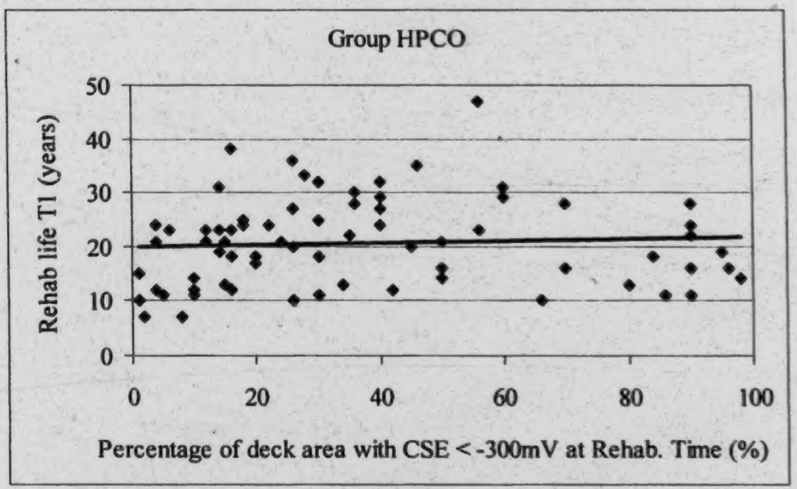

Figure 4.8 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Rehab Life T1 for HPCO Bridges

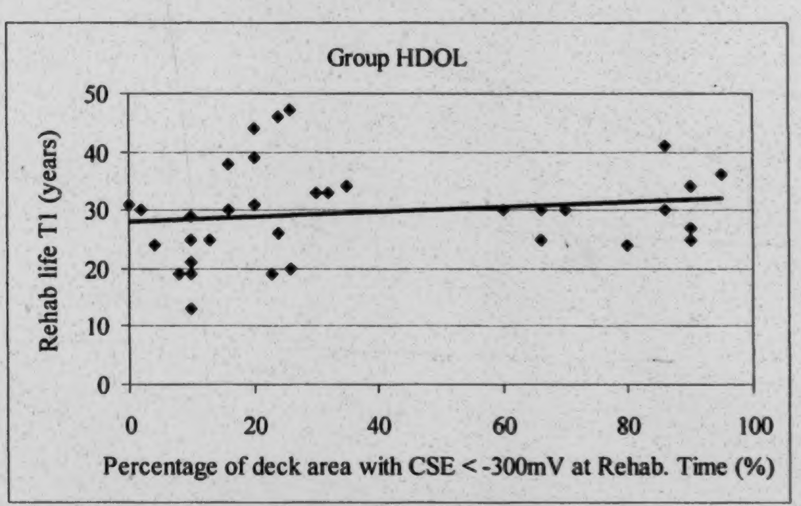

Figure 4.9 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Rehab Life T1 for HDOL Bridges

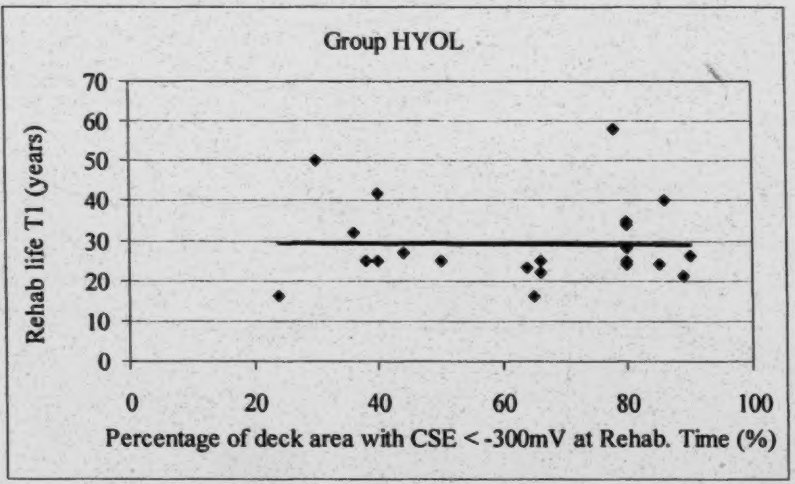

Figure 4.10 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Rehab Life T1 for HYOL Bridges

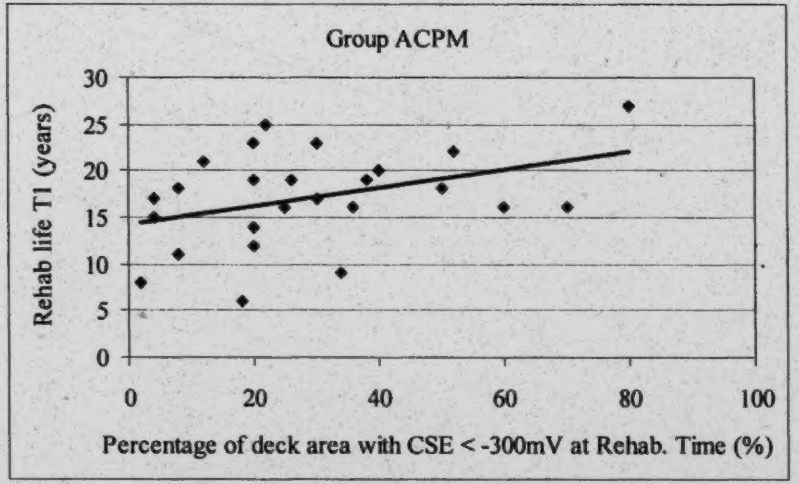

Figure 4.11 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Rehab Life T1 for ACPM Bridges

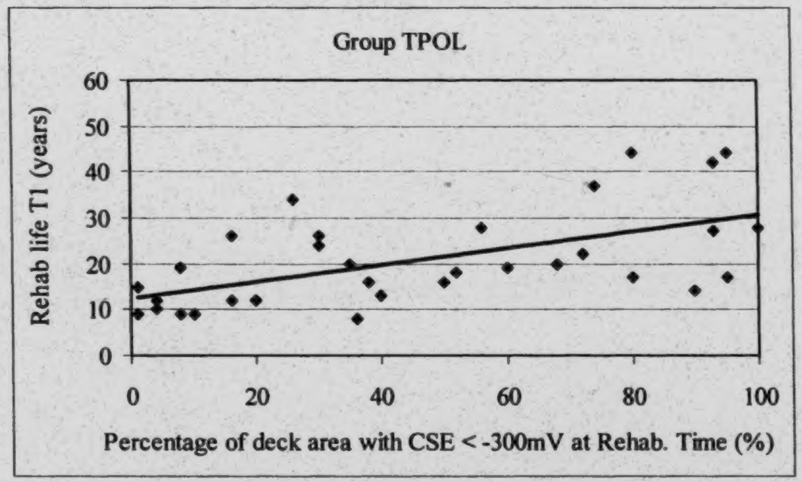

Figure 4.12 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Rehab Life T1 for TPOL Bridges

4.4.2 Impact of rehab time on life gain by rehab T2

It is expected that life gained T2 and percentage of deck area with CSE < -300mV at rehab time has an indirect proportional relationship. The linear trendline in Figure 4.13 shows an indirect proportional relationship between the percentage of deck area with CSE < -300mV at rehab time and life gained by rehab, T2, for all bridges studied in this research. The same relationship for each repair method is shown in Figures 4.14 to 4.19. A good match was found between the real relationships indicated by linear trendline in the following figures.

The slope of trendline in Figures 4.14 and 4.15 is sharper than that of Figures 4.16 to 4.18. This means that HPCO and HDOL are more sensitive to the rehabilitation time compared to the other three groups. The earlier the rehabilitation is taken, the longer the life gained by rehab T2, and the better effectiveness of the rehabilitation.

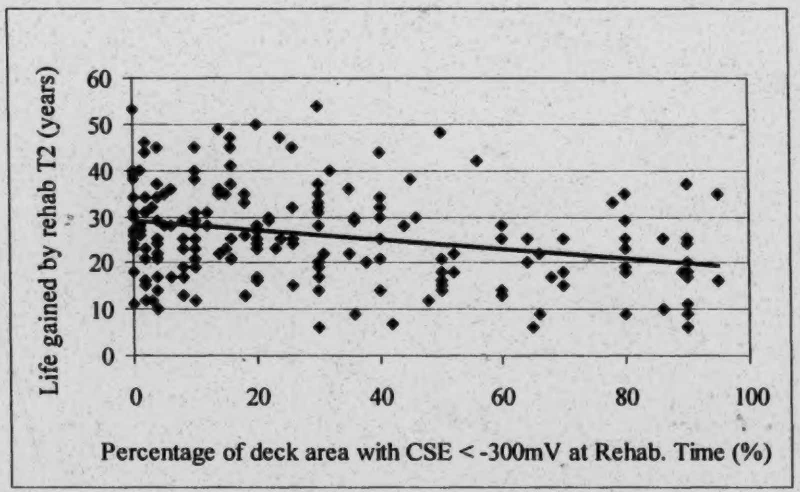

Figure 4.13 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Life Gained by Rehab T2 for All Bridges

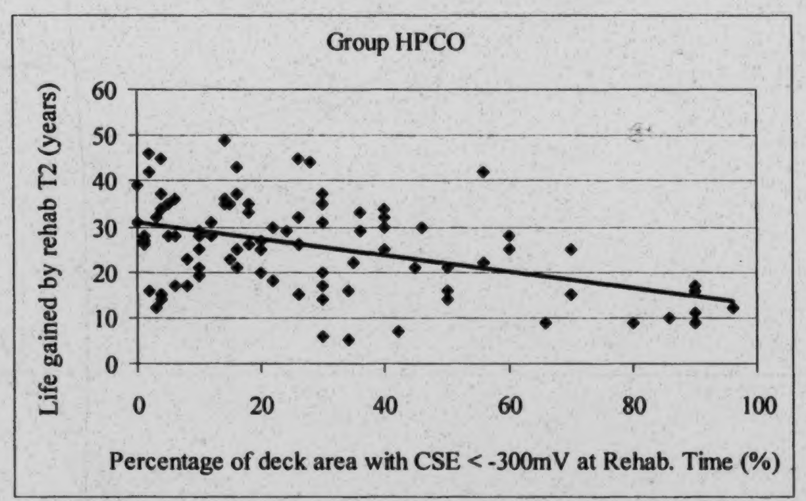

Figure 4.14 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Life Gained by Rehab T2 for HPCO Bridges

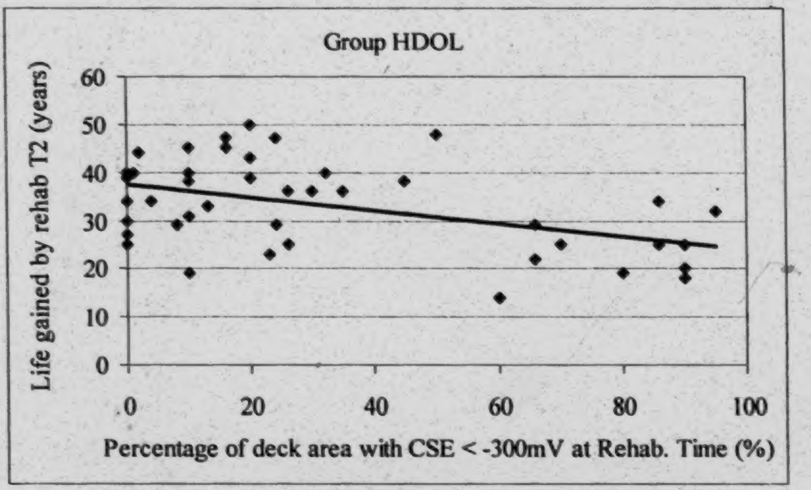

Figure 4.15 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Life Gained by Rehab T2 for HDOL Bridges

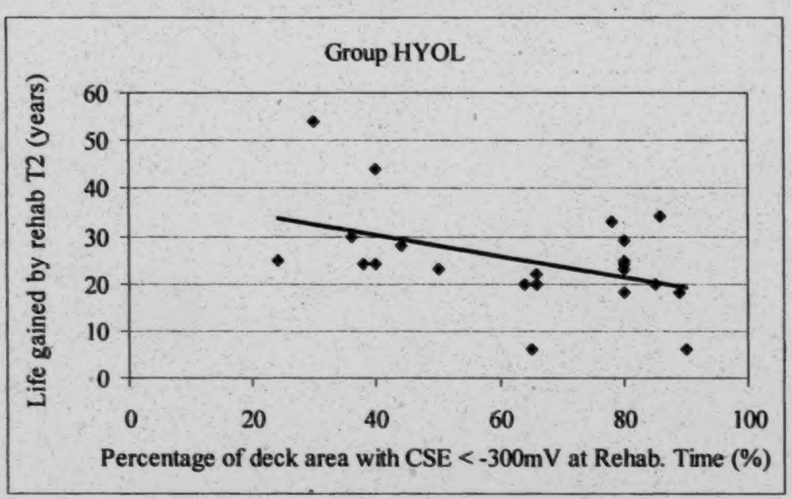

Figure 4.16 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Life Gained by Rehab T2 for HYOL Bridges

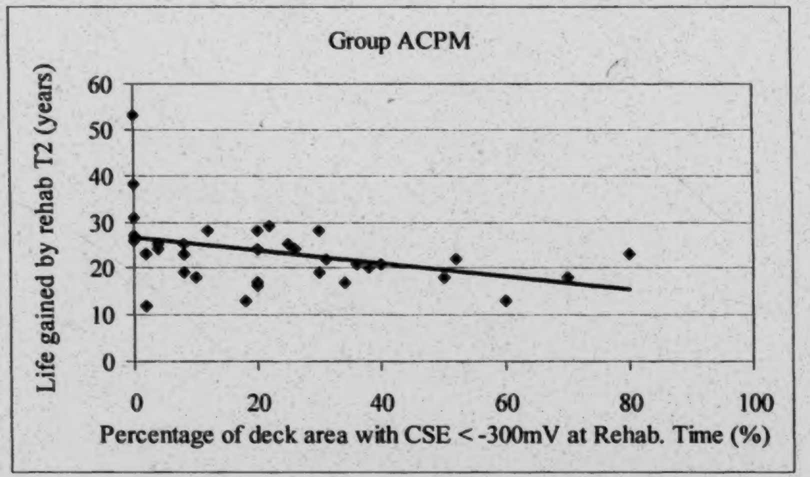

Figure 4.17 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Life Gained by Rehab T2 for ACPM Bridges

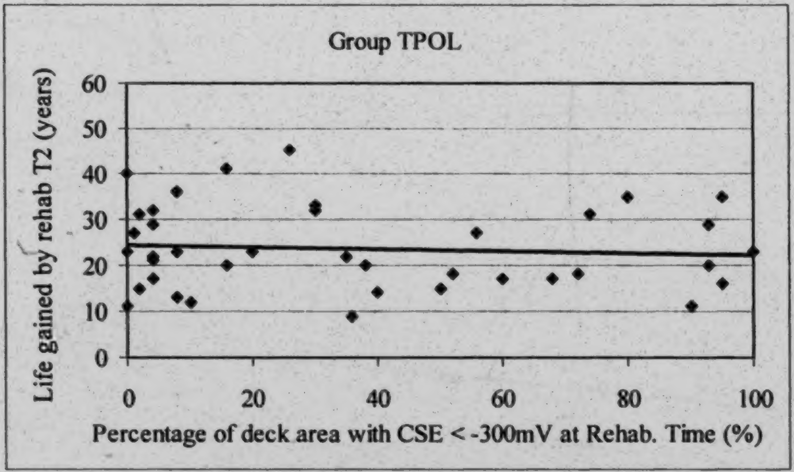

Figure 4.18 Relationship between Percentage of Deck Area with CSE < -300mV at Rehab Time and Life Gained by Rehab T2 for TPOL Bridges

4.4.3 Impact of traffic on rehab life T1

The linear trendline of Figure 4.19 shows an indirect proportional relationship between life gained T1 and traffic for all bridges. The same indirect proportional relationship is shown in Figures 4.20 and 4.22 for group HPCO and HYOL. However, no relationship is found between traffic and rehab life in Figure 4.21 for HDOL, and direct relationship is found in Figures 4.23 and 4.24 for group ACPM and TPOL. As mentioned before, the selection of upper inflation point is subjective, which may results in unexpected relationship trends.

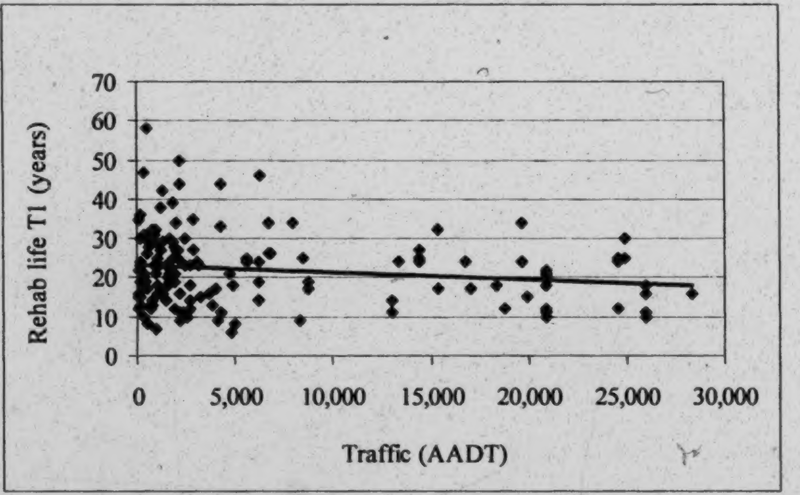

Figure 4.19 Relationship between Traffic (AADT) and Rehab Life T1 for All Bridges

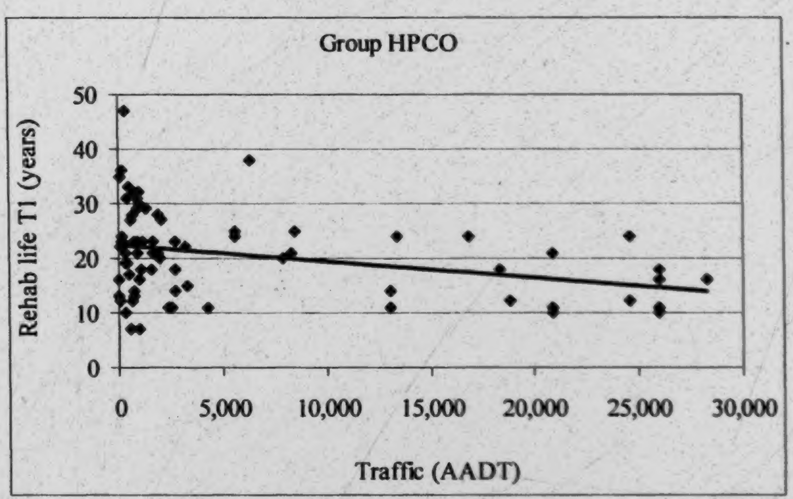

Figure 4.20 Relationship between Traffic (AADT) and Rehab Life T1 for HPCO Bridges

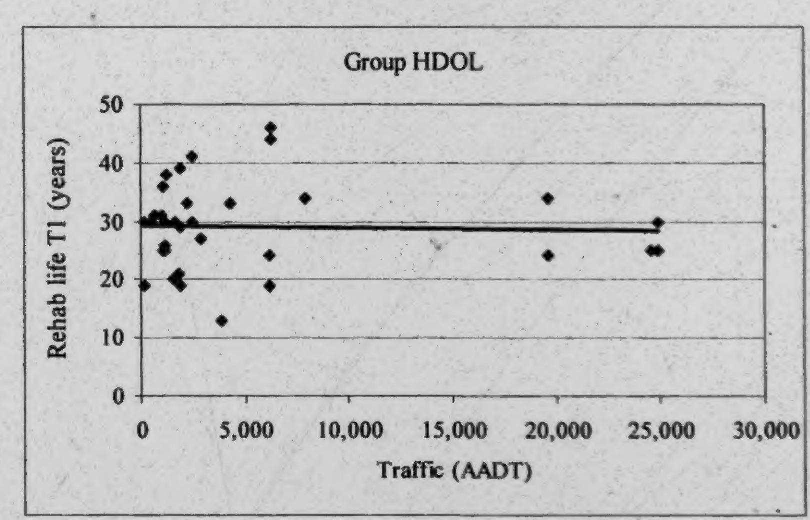

Figure 4.21 Relationship between Traffic (AADT) and Rehab Life T1 for HDOL Bridges

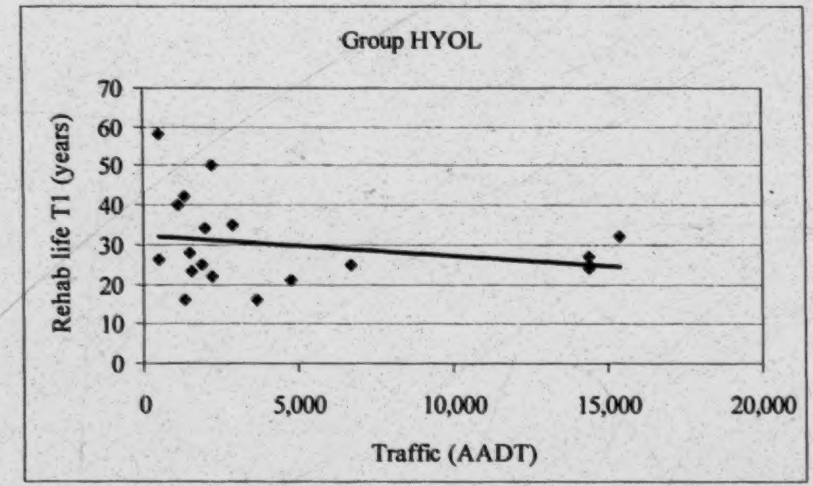

Figure 4.22 Relationship between Traffic (AADT) and Rehab Life T1 for HYOL Bridges

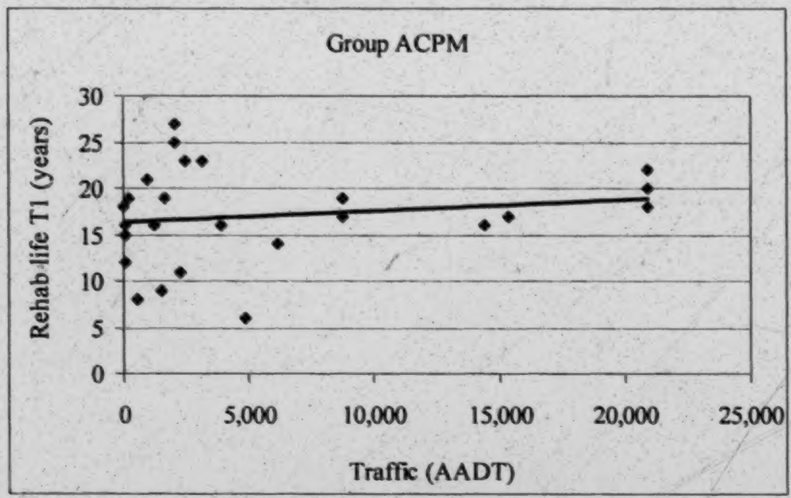

Figure 4.23 Relationship between Traffic (AADT) and Rehab Life T1 for ACPM Bridges

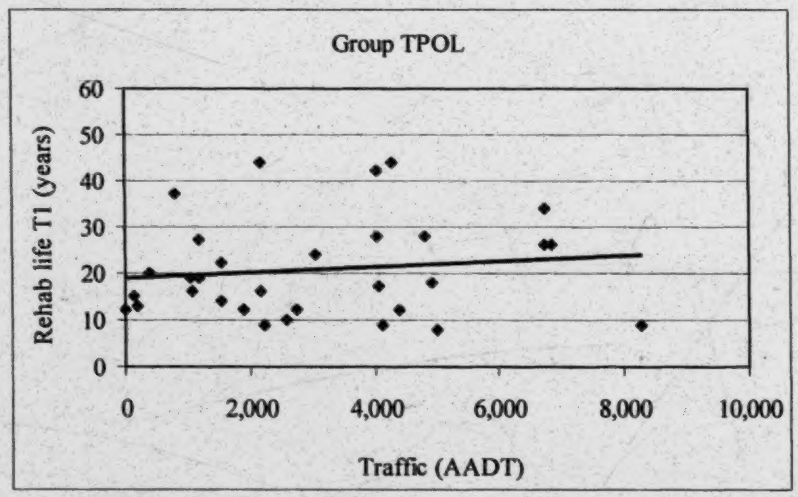

Figure 4.24 Relationship between Traffic (AADT) and Rehab Life T1 for TPOL Bridges

4.4.4 Impact of traffic on life gained by rehab T2

The linear trendline of Figure 4.25 shows an indirect proportional relationship between life gained by rehab T2 and traffic for all the bridges. The same indirect proportional relationship is shown in Figures 4.26 and 4.27 for group HPCO and HDOL. However, no relationship between traffic and life gained by rehab T2 is found in Figures 4.28 and 4.29 for bridge groups HYOL and ACPM, and a direct proportional relationship is found in Figure 4.30 for bridge group TPOL.

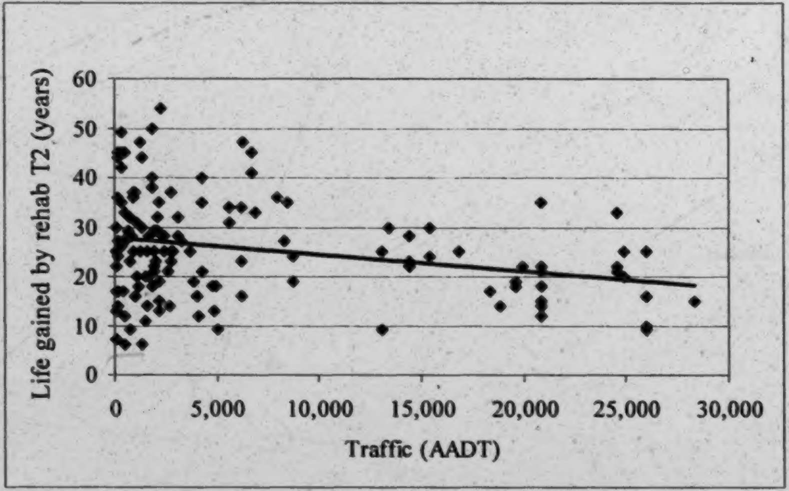

Figure 4.25 Relationship between Traffic and Life Gained by Rehab T2 for All Bridges

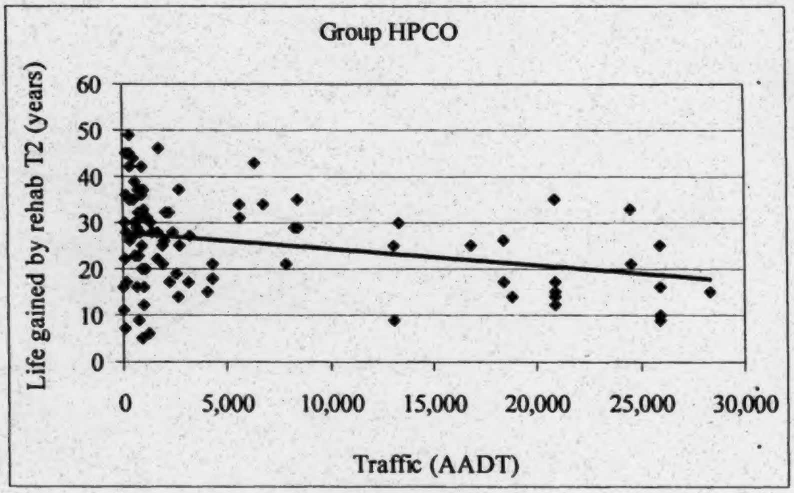

Figure 4.26 Relationship between Traffic and Life Gained by Rehab T2 for HPCO Bridges

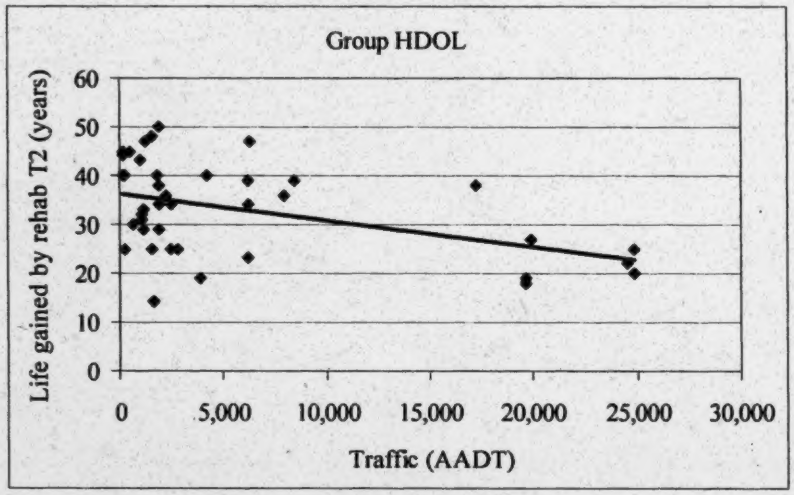

Figure 4.27 Relationship between Traffic and Life Gained by Rehab T2 for HDOL Bridges

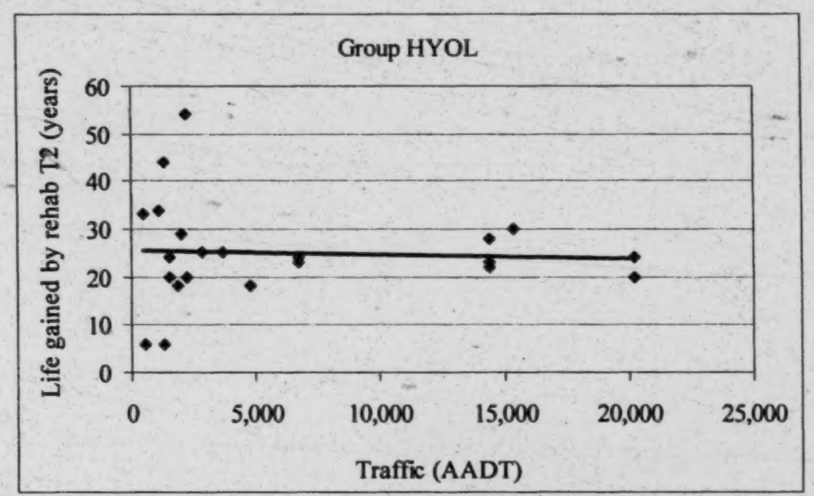

Figure 4.28 Relationship between Traffic and Life Gained by Rehab T2 for HYOL Bridges

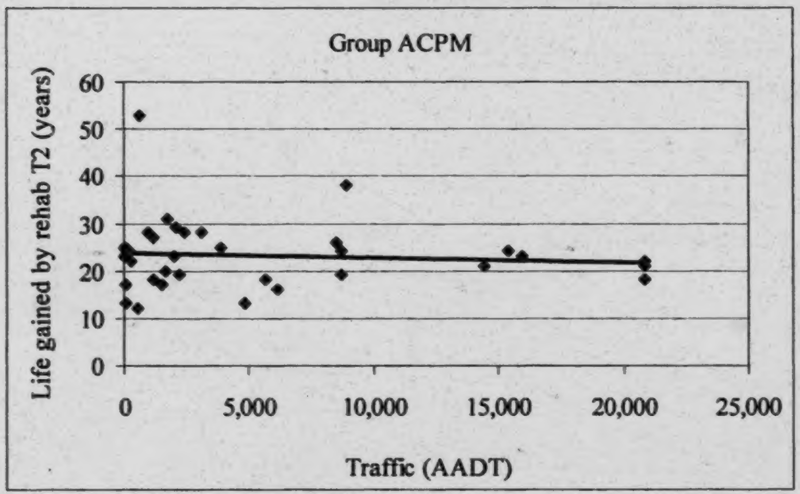

Figure 4.29 Relationship between Traffic and Life Gained by Rehab T2 for ACPM Bridges

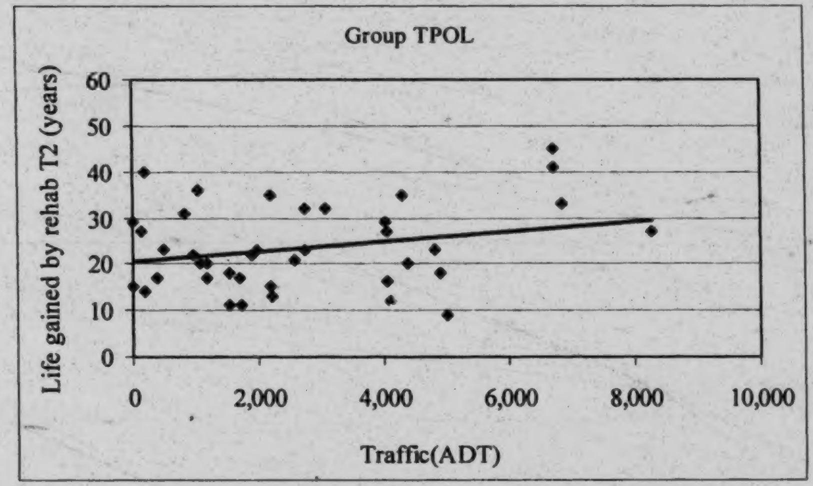

Figure 4.30 Relationship between Traffic and Life Gained by Rehab T2 for TPOL Bridges

CHAPTER 5 EFFECTIVENSS OF TWO HIGH PERFORMANCE CONCRETE BRIDGE DECK OVERLAY REHABILITATION METHODS IN ALBERTA

5.1 Introduction

In recent years, high performance concrete overlay (HPCO) is widely used in Alberta because its low permeability and its long service life. There are many types of HPCO depending on the admixture and additives added. The most commonly used admixture and additives are silica fume, fiber, fiber with silica fume, high range water-reducer, fly ash and pyrament cement. Since silica fume (SF) and fiber reinforced silica fume (FRSF) are the two main admixtures added in HPCO in Alberta. A study was conducted to evaluate the effectiveness of these two types of HPCOs.

Several bridge deck performance indicator parameters were used in this study to compare the effectiveness of different bridge deck rehabilitation overlay methods. Structural cracks are the main concern of bridge engineers, which facilitate the ingress of moisture, oxygen and chloride ions and cause the corrosion of the reinforcement steel. Hence, crack resistance performance indicators, crack length and crack density are very important in bridge deck overlay effectiveness comparisons. Other performance indicators used in this study included debonding area, bridge deck condition rating data and service life indicator such as rehab life T1 and life gained by rehab T2. Figure 5.1 shows all the performance indicators used in this study.

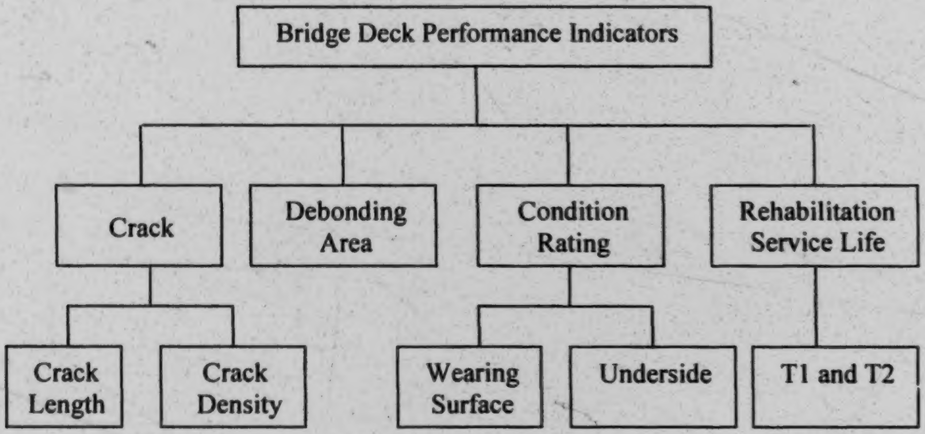

Figure 5.1 Bridge Deck Performance Indicators

5.2 Data Collection

AIT conducts bridge inspections every year, on some bridges, and the information is stored in the Bridge Deck Rehabilitation Database (BDRD). At the end of 2005, there were 154 bridge decks rehabilitated with SF and FRSF concrete bridge deck overlays. Of those 154 bridges, 30 bridges rehabilitated with SF concrete overlay and 124 bridges rehabilitated with FRSF concrete overlay. The file number of these bridges is listed in Appendix C, as well as rehabilitation history and all the performance test results.

Inventory data for overlaid bridges with SF and FRSF concrete were extracted from BDRD and are presented in Table 5.1. Bridges repaired by SF or FRSF concrete overlay have a similar average deck area, an average deck age when rehabilitated and an average overlay age. However, the average annual daily traffic and calculated cumulative traffic, since rehab of SF concrete deck overlay, were twice than those of bridges with RFSF deck overlay.

Table 5.1 Inventory Data Summary for SF and FRSF Concrete Overlays

Inventory Data	SF Concrete Overlay	FRSF Concrete Overlay
Number of Bridges with Test Data	30	124
Deck Area (m^2)		
Average	815	917
Standard Deviation	463	760
Min.	258	201
Max.	2,186	4,641
Deck Age When Rehabilitated (years)		
Average	30	26
Standard Deviation	6	6
Min.	14	10
Max.	41	46
Average Overlay Age (years, at most recent test)		
Average	5.9	6.3
Standard Deviation	2.6	3.2
Min.	1	0
Max.	11	17
Average Annual Daily Traffic		
Average	11,086	5,096
Standard Deviation	11,164	8,062
Min.	120	50
Max.	11,164	32,490
Calculated Cumulative Traffic Since Rehab		
Average	67,692	33,636
Standard Deviation	71,023	57,795
Min.	360	0
Max.	216,660	303,324

5.3 Methodology

Statistical method, t-test, was used to evaluate whether the mean values of crack length of SF and FRSF concrete bridge deck overlays were statistically different from each other. The assumption of t-test, as listed in the following passage, must be verified before using:

- (Independence): the observation of samples must be independent, which means that there is no predictable relation between sample observations;
- (Normality): The original population of samples is normally distributed. However, according to central limit theory, when the sample size is large,

for example, greater than 10 (Witte, 2004), t-test is robust for the violation of this assumption. Only when the sample is highly unlikely to be a normal distribution, a nonparametric method should be applied to test the difference of means of two groups;

- (Variances): the variance of two samples must be roughly similar. F-test could be applied to test the similarity of variance.

The null hypothesis of t-test is that there is no significant difference between two groups of performance data, while the alternative hypothesis is that there is a significant difference between the two groups of performance data.

If the normal assumption can not meet, a non-parametric test must be applied to compare the mean difference of these two groups. Wilcoxon Mann-Whitney Test is one of the most powerful of the non-parametric tests for comparing two populations. This test does not require the assumption that the differences between the two samples are normally distributed. The null hypothesis and alternative hypothesis are the same as that of a t-test. The explanation of non-parametric test can be found in statistic books written by Witte (2004).

5.4 Performance Comparison

5.4.1 Crack performance

5.4.1.1 Crack length comparison

Structural crack length is defined in meter as the total length of cracks wider than 0.3 mm in the bridge deck overlay. Structural crack length data, for overlaid bridges with SF and FRSF concrete, were extracted from BDRD and are listed in Appendix C. SF concrete bridge deck overlays have longer crack lengths than FRSF concrete bridge deck overlays, as shown in Table 5.2. However, both these two groups of data have high variance compared to the mean value. Therefore, it is necessary to conduct a statistical test to compare the mean difference of these two groups.

Table 5.2 Descriptive Statistics for Crack Length of SF and FRSF Concrete Bridge Deck Overlays

	SF Concrete Overlay		**FRSF Concrete Overlay**	
	Statistic	**Std. Error**	**Statistic**	**Std. Error**
Mean	206.53	29.83	81.10	13.76
Median	156.5		23.5	
Variance	26,690.88		20,067.07	
Std. Deviation	163.37		141.66	
Minimum	30		0	
Maximum	614		754	
Range	584		754	
Interquartile Range	190.25		78	
Skewness	1.24	0.43	2.79	0.23
Kurtosis	0.89	0.83	8.27	0.47

Because samples are selected from different groups of bridge deck overlays, the samples are independent. A box plot graph was drawn to visually verify the normality of crack length data. Figure 5.2 shows that crack length data of SF concrete overlay bridges are close to normal distribution, while crack length data of FRSF concrete bridges are unlikely to be normal distribution.

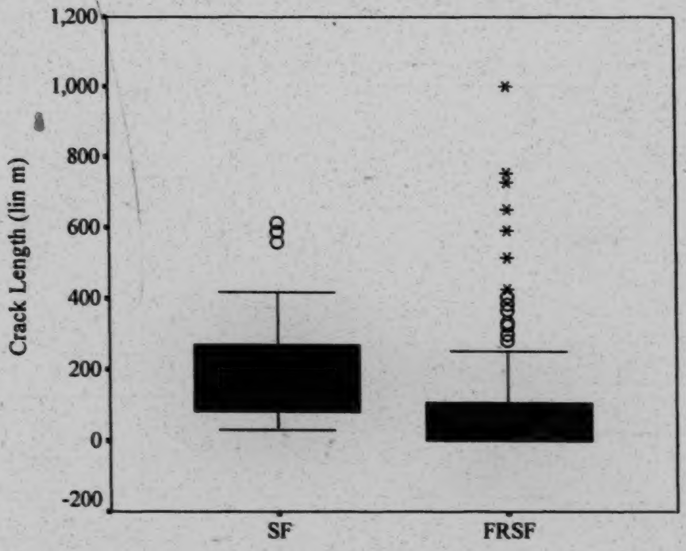

Figure 5.2 Box Plots of Crack Length Data of SF and FRSF Concrete Overlays

Square root transform of total crack length data of FRSF concrete bridge deck overlays was carried out to normalize the original data. Figures 5.3 and 5.4 are Q-Q (quantile-quantile) plots of transformed crack length data of SF and FRSF concrete bridge deck overlays, respectively. Normal Q-Q plot compares the distribution of a given variable to the normal distribution graphically. Since most data are distributed around the straight line, it is concluded that square root transformed crack length data of both groups are approximately normal distribution.

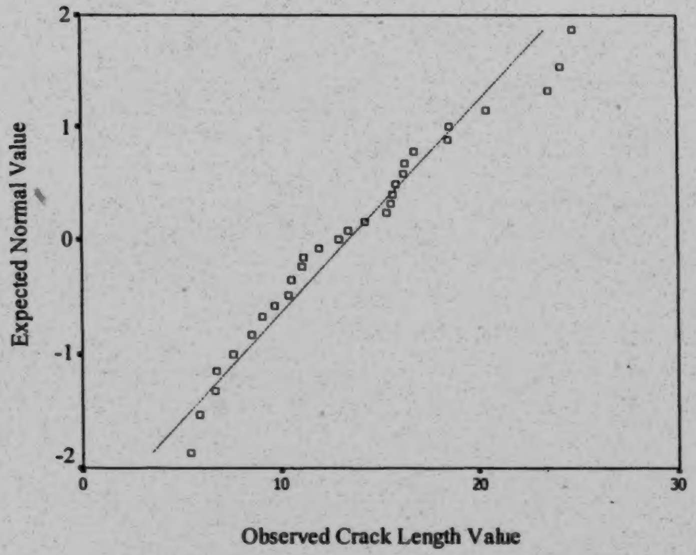

Figure 5.3 Normal Q-Q Plots of Transformed Crack Length Data for SF Concrete Bridge Deck Overlays

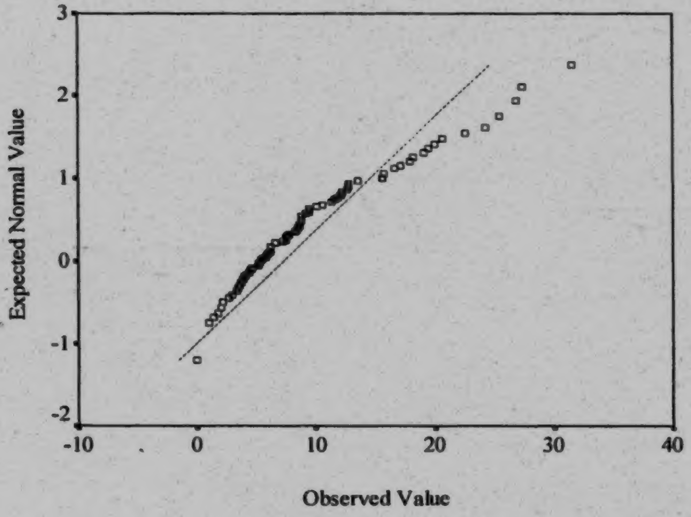

Figure 5.4 Normal Q-Q Plots of Transformed Crack Length Data for FRSF Concrete Bridge Deck Overlays

A Levene test is conducted on the transformed crack length data to compare the variance equality. The null hypothesis of the Levene test is that two groups have equal variance, while the alternative hypothesis is that two groups have unequal variance. The significance level of Levene test is 0.50, which is greater than 0.05. This means that the null hypothesis is accepted and that these two groups of data have equal variance.

Table 5.3 Levene Test on Variance Equality of Crack length Data for SF and FRSF Concrete Bridge Deck Overlays

Levene's Test for Equality of Variances		
Equal variances assumed	F	0.45
	Sig.	0.50

The null hypothesis and alternative hypothesis were explained in Section 5.3. The significance level of t-test is close to 0, as showed in Table 5.4, which is less than 0.05. This means that the null hypothesis is rejected and alternative hypothesis is accepted because there is a significant difference between the mean values of the two groups.

Table 5.4 T-test for Transformed Crack Length Data of SF and FRSF Concrete Bridge Deck Overlays

Items			**Data**
t-test for Equality of Means	t		5.45
	df		134
	Sig. (2-tailed)		2.4E-07
	Mean Difference		6.97
	Std. Error Difference		1.28
	95% Confidence Interval of the Difference	4.44	3.23
		9.51	8.93

It is concluded that FRSF concrete overlay has a better performance than SF concrete overlay in terms of crack length. The same conclusion can be drawn by observing Figure 5.2, which illustrates that 50% of SF concrete bridge deck overlays have a crack length of around 200 lin meter, while 75% of FRSF concrete bridge deck overlays have a crack length of less than 100 lin meter.

5.4.1.2 Crack density performance

The analysis of crack length data concludes that, in general, FRSF concrete overlay bridges have less crack length than SF concrete overlay bridges. However, assuming two bridges have equal crack length, a bridge with a larger area has a better crack density performance than a bridge with a small area. Hence, another crack performance indicator, crack density, is selected and calculated by dividing the crack length by bridge deck area. Structural crack length and bridge deck area data are extracted from Bridge Deck Rehabilitation Database, and are listed in Appendix C.

As shown in Table 5.5, SF concrete bridge deck overlays have higher crack density than FRSF concrete bridge deck overlays. Although the variances of both groups are small compared to the mean value, a statistical test was conducted to compare the statistical mean difference of these two groups.

Table 5.5 Descriptive Statistics on Crack Density Data of SF and FRSF Concrete Bridge Deck Overlays

	SF Concrete Overlay		**FRSF Concrete Overlay**	
	Statistic	**Std. Error**	**Statistic**	**Std. Error**
Mean	0.25	0.02	0.08	0.01
Median	0.25		0.03	
Variance	0.02		0.01	
Std. Deviation	0.12		0.10	
Minimum	0.05		0	
Maximum	0.53		0.41	
Range	0.48		0.41	
Interquartile Range	0.19		0.10	
Skewness	0.26	0.43	1.66	0.23
Kurtosis	-0.34	0.83	1.77	0.46

Since samples are selected from different groups of bridge deck overlays, they are independent with each other. A box plot graph is drawn to visually verify the normality assumption. Figure 5.5 shows that crack density of SF concrete overlay bridges is close to normal distribution, while crack density data of FRSF concrete overlay bridges are highly unlikely to be normal distribution.

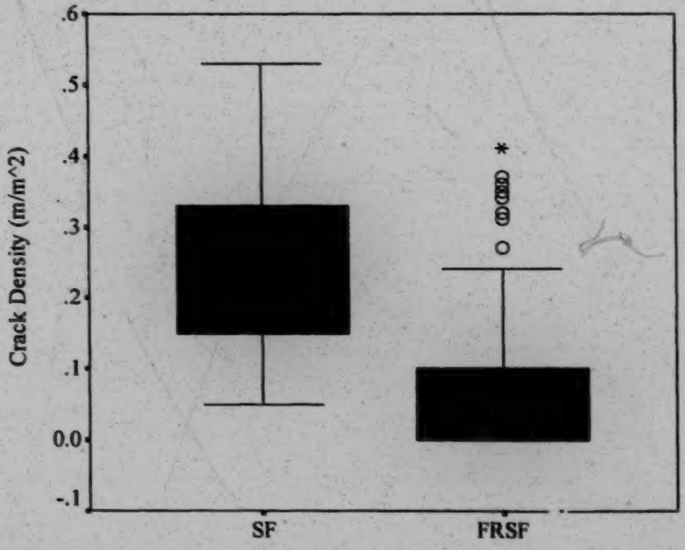

Figure 5.5 Box Plots of Crack Density Data for SF and FRSF Concrete Bridge Deck Overlays

Various data transforms have been carried out to normalize the crack density data of FRSF concrete bridge deck overlays. However, none of the data transformed has been successful. Hence, a non-parametric test is applied to examine the mean difference of these two samples.

The null hypothesis and alternative hypothesis were explained in Section 5.3. The significance level of the Mann-Whitney U test was 5E-10, as showed in Table 5.6, which was much less than 0.05. This means that the null hypothesis is rejected and alternative hypothesis is accepted because there is a significant difference between the mean values of the two groups. Since mean crack density value of FR bridge deck overlays is greater than that of FRSF concrete bridge deck overlays, it is concluded that FRSF concrete bridge deck overlays have better performance than SF concrete bridge deck overlays in terms of crack density.

Table 5.6 Non-parametric Test of Mean Difference for Crack Density Data of SF and FRSF Concrete Bridge Deck Overlays

Ranks

Type	N	Mean Rank	Sum of Ranks
SF	30	111.7	3351
FRSF	111	60	6660
Total	141		

Test Statistics (a)

	Value
Mann-Whitney U	444
Wilcoxon W	6660
Z	-6.21
Asymp. Sig. (2-tailed)	5E-10

A Grouping Variable: FRSF1

5.4.2 Debonding performance

Debonding is defined by RILEM (International Union of Laboratories and Experts in Construction Materials, systems, and structures) as "a gradual process where slow growth of cracks occurs at the interface. The characteristics of such growth, including the coalescence of cracks, are controlled by fracture-mechanics based parameters." Fracture-based characterization of the interface will produce more useful indicators of interfacial quality than the ultimate bond strength alone and represent a better interface of damage and durability.

Chain drag can detect concrete bridge deck delamination or debonding based on the sound produced. For overlaid bridge decks, the deep-pitched sounds indicate debonding. Therefore, a chain drag test result is adopted as a bridge deck overlay debonding performance indicator.

As shown in Table 5.7, SF concrete bridge deck overlays have a higher debonding area than FRSF concrete bridge deck overlays. However, the mean value of both overlays are so close to each other that it is necessary to conduct statistical tests in order to compare the statistic mean difference of these two groups.

Table 5.7 Descriptive Statistics on Chain Drag Test Data of SF and FRSF Concrete Bridge Deck Overlays

	SF Concrete Overlays		**FRSF Concrete Overlays**	
	Statistic	**Std. Error**	**Statistic**	**Std. Error**
Mean	0.65%	0.18%	0.63%	0.10%
Median	0%		0%	
Variance	0.01%		0.01%	
Std. Deviation	0.99%		1.04%	
Minimum	0		0	
Maximum	3.5%		4.1%	
Range	3.5%		4.1%	
Interquartile Range	1.1%		1.0%	
Skewness	1.57	0.43	1.84	0.22
Kurtosis	1.68	0.83	2.66	0.45

Since samples are selected from different groups of bridge deck overlays, the samples are independent. A box plot graph is drawn to check the normality assumption visually. Figure 5.6 shows that chain drag data of both SF and FRSF concrete bridge deck overlays are unlikely to be normally distributed. Hence, a non-parametric test is applied to test the mean difference of these two samples.

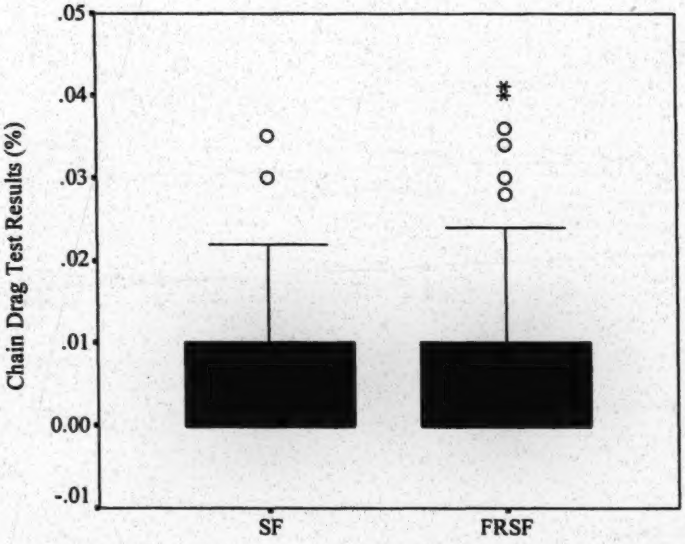

Figure 5.6 Box Plots of Chain Drag Data for SF and FRSF Concrete Bridge Deck Overlays

The null hypothesis and alternative hypothesis were explained in Section 5.3. The significance level of non-parametric test of 0.81, as showed in Table 5.8, is much greater than 0.05, which means that the null hypothesis is accepted because there is no significant difference between the mean values of the two groups. It is then concluded that, in terms of debonding performance, FRSF and SF concrete bridge deck overlays have the same performance.

Table 5.8 Non-parametric Test on Mean Difference of Debonding Data of SF and FRSF Concrete Bridge Deck Overlays

Ranks

Bridge Deck Overlay Types	N	Mean Rank	Sum of Ranks
SF	30	75.02	2250.5
FRSF	116	73.11	8480.5
Total	146		

Test Statistics (a)

	Data
Mann-Whitney U	1694.5
Wilcoxon W	8480.5
Z	-0.24
Asymp. Sig. (2-tailed)	0.81

A Grouping Variable: FRSF1

5.4.3 Condition rating performance

Condition rating test is carried out by Alberta's bridge expertise, according to the specifications listed in Table 5.9. Condition rating data are based on a scale of 1-9, where 9 means the bridge deck overlay is in very good condition and 1 means bridge deck overlay needs to be repaired immediately. In Alberta, ratings are normally grouped into the following six categories: 9 to 7, 6 and 5, 4, 3, 2, 1. The classification of condition ratings is explained in Table 5.9.

Table 5.9 Definition of Bridge Deck Condition Rating in Alberta

Rating	**Classification**	**Condition Descriptions**
9	Very Good	New condition.
8		Almost new condition.
7	Good	Could be upgraded to new condition with very little effort.
6		Generally good condition. Functioning as designed with no signs of distress of deterioration.
5	Adequate	Acceptable condition. Minor flaws, but functioning as intended.
4		Below minimum acceptable condition.
3	Poor	Presence of distress or deterioration. Not functioning as intended.
2		May require continued observation until work is completed.
1	Immediate Action	Danger of collapse and/or danger to users.

All condition rating data for bridges overlaid with SF and FRSF concrete were extracted from BDRD and the results are listed in Appendix C.

5.4.3.1 Wearing Surface Condition Rating Data Comparison

Table 5.10 shows that FRSF concrete bridge deck overlays have higher wearing surface condition ratings than SF concrete bridge deck overlays. However, the variances of both overlays are great compared to the mean values. Therefore, it is necessary to conduct a statistical test to compare the statistic mean difference of these two groups.

Table 5.10 Descriptive Statistics on Bridge Deck Wearing Surface Condition Rating Data of SF and FRSF Concrete Bridge Deck Overlays

	SF Concrete Overlays		**FRSF Concrete Overlays**	
	Statistic	**Std. Error**	**Statistic**	**Std. Error**
Mean	6.37	0.22	6.63	0.12
Median	6.15		6.8	
Variance	1.52		1.61	
Std. Deviation	1.23		1.27	
Minimum	4		3	
Maximum	8		8	
Range	4		5	
Interquartile Range	2		2.5	
Skewness	-0.12	0.43	-0.22	0.22
Kurtosis	-1.12	0.83	-1.29	0.44

Since samples were selected from different groups of bridge deck overlays, the samples are independent with each other. A box plot graph is drawn to check the normality assumption visually. Figure 5.7 shows that wearing surface condition rating data of SF concrete overlay bridges are close to normal distribution, while the data of FRSF concrete overlay bridges are unlikely to be normal distribution.

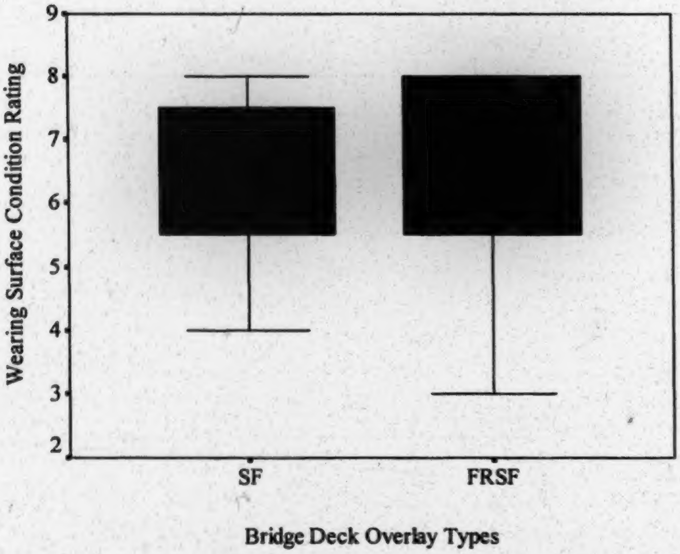

Figure 5.7 Box Plot of Bridge Deck Wearing Surface Condition Rating for SF and FRSF Concrete Overlays

Various data transforms have been carried out to normalize the wearing surface condition rating data of FRSF concrete bridge deck overlays. However, none of the data transforms have been successful. Hence, a non-parametric test is applied to test the mean difference of these two samples.

The null hypothesis and alternative hypothesis were explained in Section 5.3. The significance level of Mann-Whitney is 0.3, as showed in Table 5.11, which is much greater than 0.05. This means that the null hypothesis is accepted because there is no significant difference between the mean values of the two groups. It is concluded that FRSF and SF concrete bridge deck overlays have the same performance regarding bridge deck wearing surface condition ratings.

Table 5.11 Non-parametric Test of Mean Difference for Wearing Surface Condition Rating Data of SF and FRSF Concrete Bridge Deck Overlays

Bridge Deck Overlay Types	N	Mean Rank	Sum of Ranks
SF	30	67.97	2039
FRSF	119	76.77	9136
Total	149		

Test Statistics (a)

	Data
Mann-Whitney U	1574
Wilcoxon W	2039
Z	-1.03
Asymp. Sig. (2-tailed)	0.30

A Grouping Variable: FRSF1

5.4.3.2 Bridge Deck Underside Condition Rating Data Comparison

Bridge deck underside condition rating data are also based on the scale of 1-9, as explained in Table 5.9. Table 5.12 shows that FRSF concrete bridge deck overlays have higher underside condition ratings than SF concrete bridge deck overlays. However, the variances of both overlays are high compared to the mean values. Therefore, it is necessary to conduct a statistical test to compare the statistic mean difference of these two groups.

Table 5.12 Descriptive Statistics on Underside Condition Rating Data of SF and FRSF Concrete Bridge Deck Overlays

	SF		FRSF	
	Statistic	Std. Error	Statistic	Std. Error
Mean	6.40	0.28	6.64	0.12
Median	5.5		6.35	
Variance	1.61		1.57	
Std. Deviation	1.27		1.25	
Minimum	4.8		4	
Maximum	8		8	
Range	3.2		4	
Interquartile Range	2.45		2.5	
Skewness	0.48	0.50	-0.02	0.23
Kurtosis	-1.86	0.97	-1.83	0.47

Since samples are selected from different groups of bridge deck overlays, the samples are independent of each other. A box plot graph is drawn to visually verify the normality assumption. Figure 5.8 shows that underside condition rating data of both SF and FRSF concrete bridge deck overlays are unlikely to be normal distribution. Hence, a non-parametric test is applied to test the mean difference of these two samples.

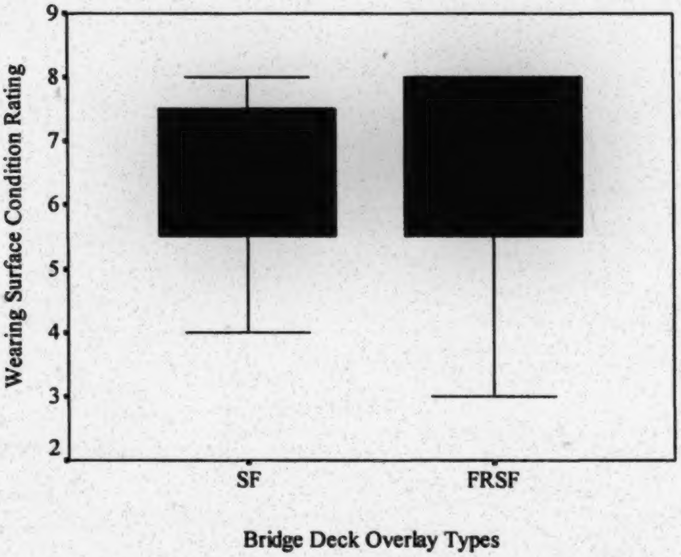

Figure 5.8 Box Plot of Bridge Deck Underside Condition Rating for SF and FRSF Concrete Overlays

The null hypothesis and alternative hypothesis were explained in Section 5.3. The significance level of a non-parametric test of 0.56, as showed in Table 5.13, is much greater than 0.05, which means that null hypothesis is accepted because there is no significant difference between mean values of two groups. It is then concluded that FRSF and SF concrete bridge deck overlays have the same performance regarding the underside condition rating.

Table 5.13 Non-parametric Test of Mean Difference for Underside Condition Rating Data of SF and FRSF Concrete Bridge Deck Overlays

Ranks

Bridge Deck Overlay Types	N	Mean Rank	Sum of Ranks
SF	21	59.93	1258.5
FRSF	106	64.81	6869.5
Total	127		

Test Statistics (a)

	Data
Mann-Whitney U	1027.5
Wilcoxon W	1258.5
Z	-0.58
Asymp. Sig. (2-tailed)	0.56

A Grouping Variable: FRSF1

5.4.4 Rehabilitation service life performance

For bridges that have never been repaired, the rehabilitation service life is the bridge age at time of bridge deck rehabilitation. A long rehabilitation service life means a bridge has a good deck performance rating. For repaired bridges, rehabilitation service life is defined by Earth Tech (ET) model as the time period from the time of rehab to the time when CSE returns to the 50% of deck area with CSE < -300mV level. Rehabilitation service life is represented by T2, i.e. life gained by rehabilitation. All T2 data are extracted the same way as explained in Section 3.4, from concrete deck rehabilitation prediction curve by ET model, and all the results are listed in Appendix C.

As shown in Table 5.14, FRSF concrete bridge deck overlays have longer rehabilitation service lives than SF concrete bridge deck overlays. However, the variance of FRSF concrete bridges is much higher than that of SF concrete overlay bridges. Therefore, it is necessary to conduct a statistical test to compare the mean difference of these two groups.

Table 5.14 Descriptive Statistics on T2 Data for SF and FRSF Concrete Bridge Deck Overlays

	SF		FRSF	
	Statistic	**Std. Error**	**Statistic**	**Std. Error**
Mean	17.42	1.76	27.40	1.18
Median	16.5		28	
Variance	37.17		96.97	
Std. Deviation	6.10		9.85	
Minimum	9		5	
Maximum	28		49	
Range	19		44	
Interquartile Range	10.5		14.0	
Skewness	0.52	0.64	-0.23	0.29
Kurtosis	-0.59	1.23	-0.15	0.57

Because samples are selected from two different groups of bridge deck overlays, the samples are independent of each other. A box plot graph is drawn to visually verify the normality assumption. Figure 5.9 shows that crack density data of both SF and FRSF concrete bridge deck overlays are close to normal distribution.

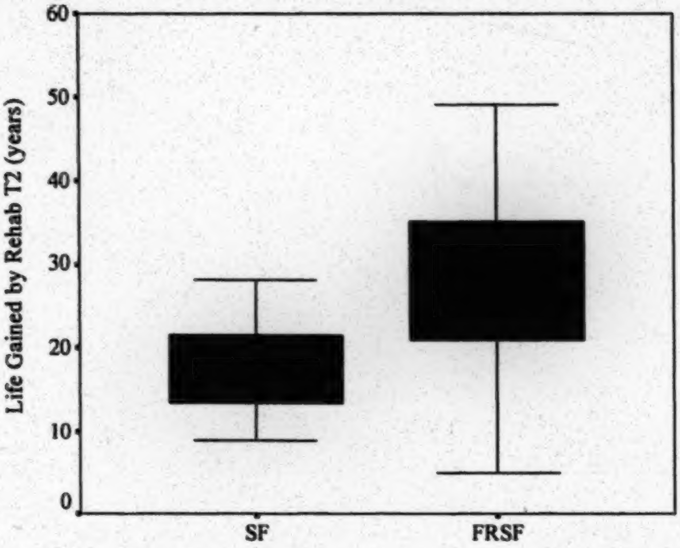

Figure 5.9 Box Plots of T2 Data for SF and FRSF Concrete Overlays

Levene test was conducted to compare the variance equality of these two groups. The null hypothesis of the Levene test is that two groups have equal variance, while the alternative hypothesis is that two groups have unequal variance. The significance level of the Levene test is 0.08, which is greater than 0.05. This means that null hypothesis is accepted because these two groups of data have equal variance.

Table 5.15 Levene Test Equality of Variance of T2 Data for SF and FRSF Concrete Bridge Deck Overlays

Levene's Test for Equality of Variances	
F	3.06
Sig.	0.08

The null hypothesis and alternative hypothesis were explained in Section 5.3. The significance level of t test of 0.001, as showed in Table 5.16, is less than 0.05, which means that null hypothesis is rejected and alternative hypothesis is accepted because there is a significant difference between the mean values of the two groups.

Table 5.16 T-test on Mean Difference of T2 data for SF and FRSF Concrete Bridge Deck Overlays

Items			**Data**
t-test for Equality of Means	t		-3.39
	df		80
	Sig. (2-tailed)		0.001
	Mean Difference		-9.98
	Std. Error Difference		2.94
	95% Confidence Interval of the Difference	-15.84	-15.84
		-4.13	-4.13

It is concluded that a FRSF concrete bridge deck overlay has a longer rehabilitation service life than a SF concrete bridge deck overlay. A box plot graph, Figure 5.5, also clearly shows that the average T2 value of a FRSF concrete bridge deck overlay is much higher than that of a SF concrete bridge deck overlay.

Most FRSF concrete bridge deck overlays have the T2 value of 30 years, while most SF concrete bridge deck overlays have T2 value of around 20 years.

5.5 Impact of Factors on Performance

As discussed in Section 4.2.3, factors such as bridge age, aggregate type, average daily traffic, environmental factors and construction materials have a great impact on the effectiveness of a bridge deck overlay. Section 5.5.1 and Section 5.5.2 will discuss the impact of traffic volume and bridge deck age on two bridge deck overlay methods, SF and FRSF concrete, respectively.

5.5.1 Impact of traffic on the bridge deck overlay performance .

Figure 5.10 and Figure 5.11 show a direct proportional relationship between traffic and total bridge deck crack length and crack density for both SF and RFSF concrete overlays. Total crack length and crack density increase with the increase of traffic for SF concrete bridge deck overlays. However, the correlation value, 0.078 and 0.01, is very low. It is difficult to draw a conclusion that traffic has a great impact on crack length and crack density. On the other hand, no relationship was found for FRSF concrete bridge deck overlay.

Several reasons could possibly explain these relationships. Firstly, some hidden factors, which may cause the decrease of crack length, counteract the impact of traffic. Secondly, most traffic data are concentrated on the range of less than 5,000 vehicles per day. Had the data been existed for a wider range, the correlation could have been much higher.

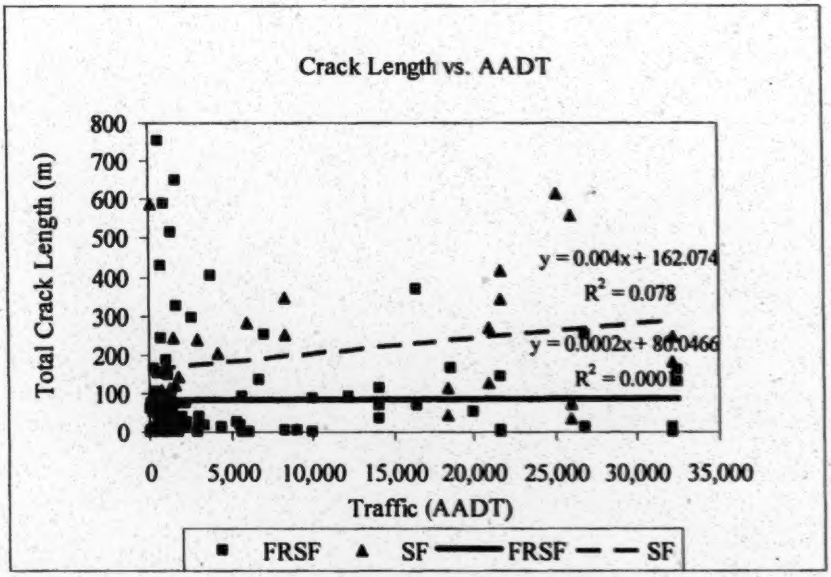

Figure 5.10 Relationship between Crack Length Data and Average Annual Daily Traffic

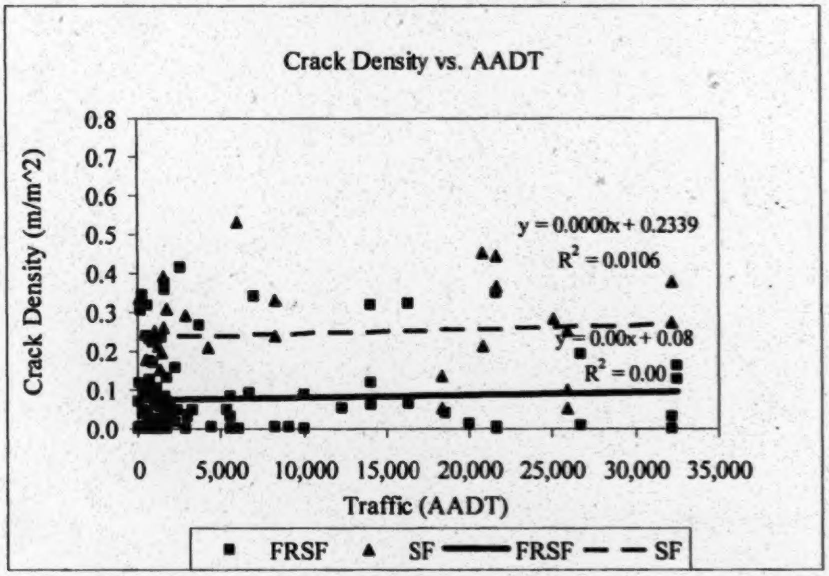

Figure 5.11 Relationship between Crack Density and Average Annual Daily Traffic

Figure 5.12 presents inverse proportional relationship between traffic and condition rating. It shows a decrease on the bridge deck wearing surface condition rating with an increase of traffic for SF concrete bridge deck overlay. However, the correlation is low, and as a result, it is difficult to accept that traffic has a great impact on condition rating. No relationship has been found for FRSF concrete bridge deck overlay from Figures 5.12 and 5.13.

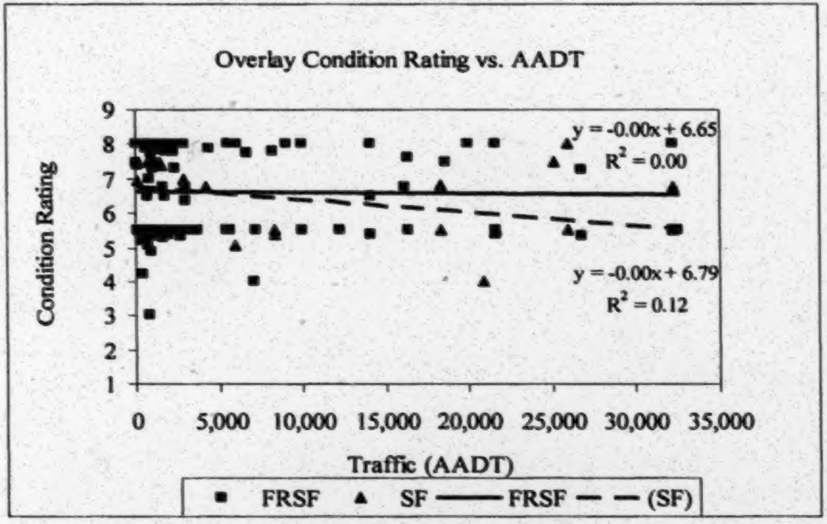

Figure 5.12 Relationship between Bridge Deck Wearing Surface Condition Rating and Average Annual Daily Traffic

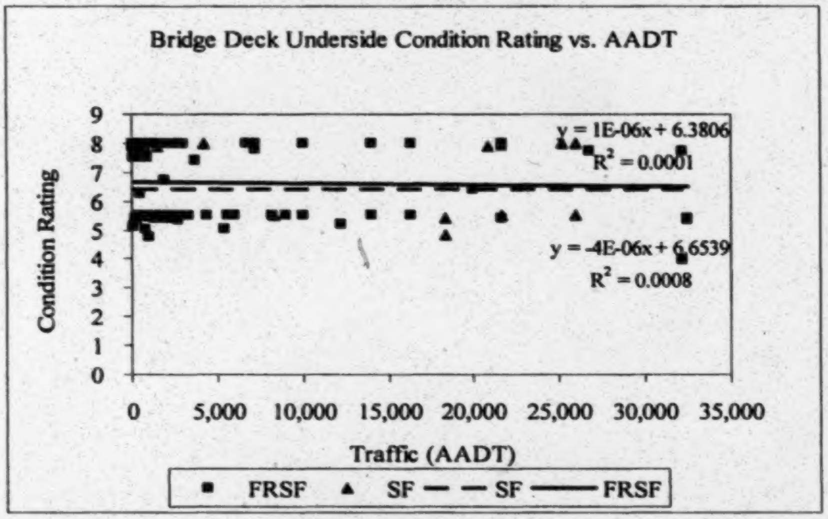

Figure 5.13 Relationship between Bridge Deck Underside Condition Rating and Average Annual Daily Traffic

5.5.2 Impact of overlay age on bridge deck overlay performance

Both Figures 5.14 and 5.15 show an increase in total crack length and crack density over time for FRSF concrete bridge deck overlays. The correlations in Figures 5.14 and 5.15 are 0.05 and 0.06, respectively. No relationship was found between crack length and time for SF concrete bridge deck overlay, as well as crack density and time.

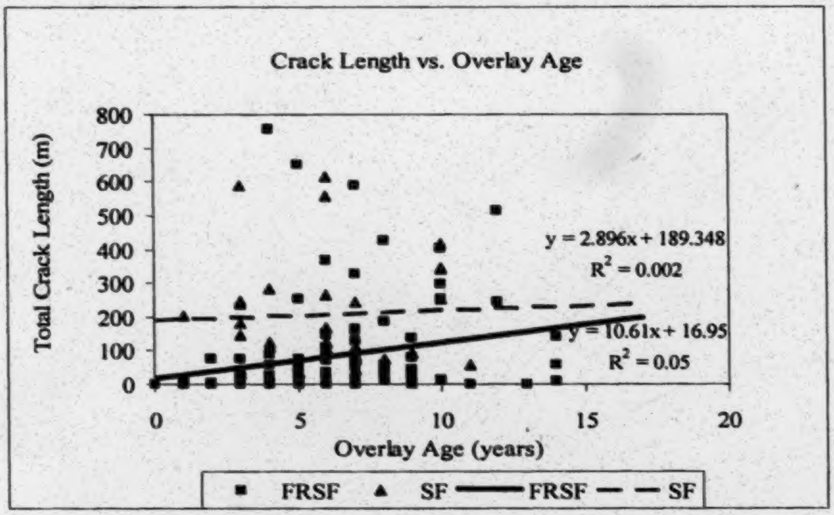

Figure 5.14 Relationship between Total Crack Length and Overlay Age

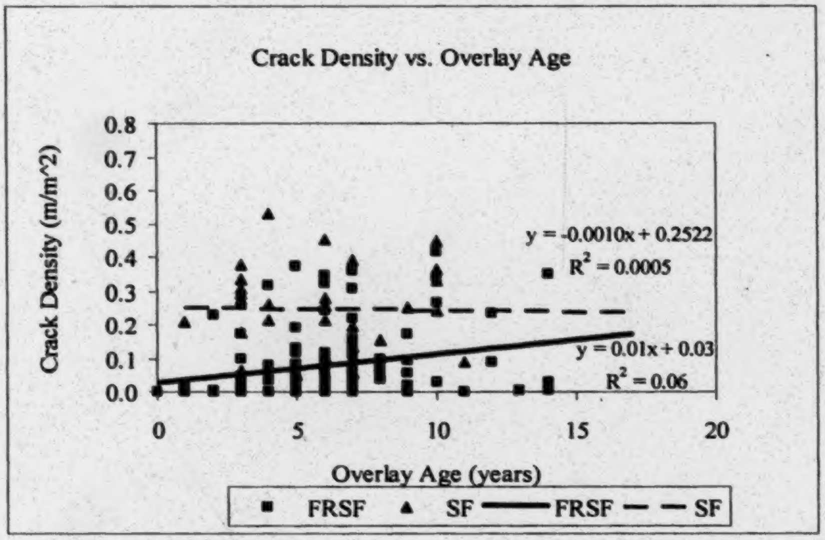

Figure 5.15 Relationship between Crack Density and Overlay Age

Figure 5.16 shows a decrease of bridge deck wearing surface condition rating data, over time for both SF and FRSF concrete bridge deck overlays. The correlations are 0.10 and 0.06, respectively. Figure 5.17 shows a decrease in bridge deck underside level 2 rating data over time, for SF concrete bridge deck overlay. With the correlation of 0.397, it is easy to accept that overlay age has a great impact on bridge deck underside condition rating for SF concrete bridge deck overlay. No relationship was found for FRSF concrete bridge deck overlay in Figure 5.17.

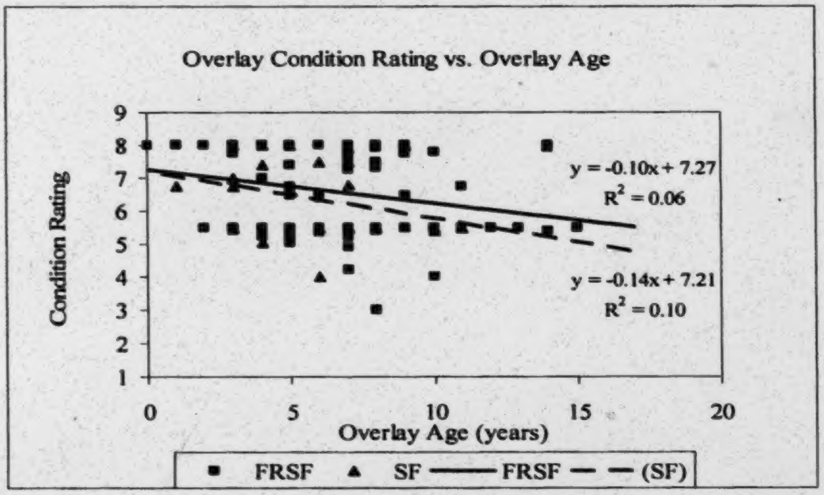

Figure 5.16 Relationship between Bridge Deck Surface Condition Rating Data and Overlay Age

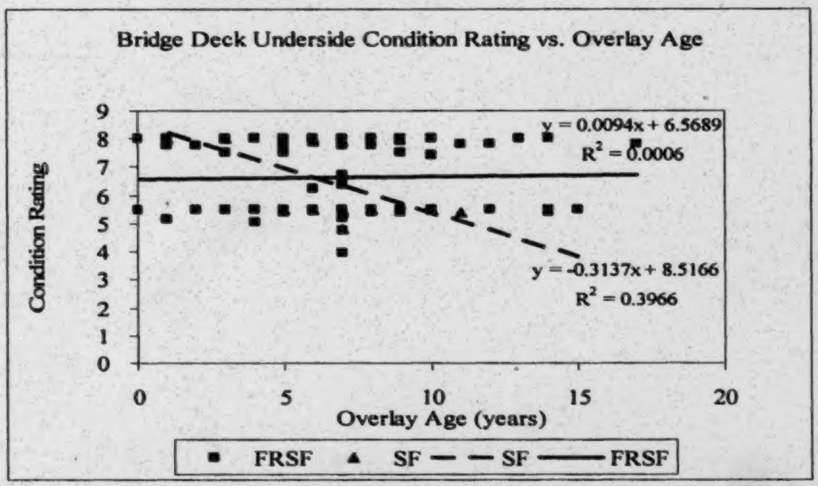

Figure 5.17 Relationship between Bridge Deck Underside Condition Rating Data and Overlay Age

5.6 Summary

This chapter compares the effectiveness of SF and FRSF concrete bridge deck overlays by the following four performance indicators: cracks, debonding, condition ratings and rehabilitation service life. Table 5.17 summarizes all the performance data for SF and FRSF concrete bridge deck overlays:

Table 5.17 Performance Data Summary

Performance Data	**SF Concrete Overlay**	**FRSF Concrete Overlay**	**Test Method**	**Mean Different**
Cracks (> 0.3 mm width)				
Average Total Cracks Length (lin m)	207	81.10	t-test	Yes
Std Dev of Total Cracks (lin m)	163	141	-	-
Average Crack Density (lin m/m^2)	0.25	0.08	Non-parametric	Yes
Std Dev of Crack Density (lin m/m^2)	0.12	0.10	-	-
Debonding				
Average Debonding (% of Overlay)	0.65	0.63	Non-parametric	No
Std Dev of Debonding (% of Overlay)	0.99	1.03	-	-
Condition Rating (9 = excellent; 1 = very bad)				
Average Wearing Surface Rating	6.37	6.63	Non-parametric	No
Std Dev of Wearing Surface Rating	1.23	1.27	-	-
Average Deck Underside Rating	6.40	6.64	Non-parametric	No
Std Dev of Underside Rating	1.27	1.25	-	-
Rehabilitation Service Life				
Corrosion Life Gained (years)	17.4	27.4	t-test	Yes
Std Dev of Corrosion Life Gained	6.10	9.8	-	-

Note: Mark "-"indicates data is not applicable.

The following conclusions can be drawn from statistical comparisons between SF and FRSF concrete overlay performance data:

1. FRSF concrete overlay bridges have shown better performances than SF concrete overlay bridges in terms of total crack length and crack density. FRSF concrete overlay bridges have 61% less average crack length and 68% less average crack density than SF concrete overlay bridges;

2. Traffic has a greater impact on crack performance of SF concrete overlay bridges than that of FRSF concrete overlay bridges. Crack length and crack density of SF concrete overlay bridges increase with the increase of traffic. However, for FRSF concrete overlay bridges, crack length and crack density remain on the same level, even when traffic increases;

3. There is a significant difference between SF and FRSF concrete overlay bridges in corrosion performance according to life gained by rehab. FRSF concrete overlay bridges have average corrosion life gained by rehab of 28 years, which is about 55% higher than the life gained by rehab of SF concrete overlay bridges;

4. Although an average wearing surface level 2 rating data and deck underside level 2 rating data of RFSF bridges are higher than that of SF concrete overlay bridges, the result of t-tests shows that, statistically, there is no significant difference between these two values. In other words, FRSF concrete overlay bridges and SF concrete overlay bridges have similar visual rating performance;

5. Structural crack length and crack density of SF and FRSF concrete overlay bridge deck overlays increase overtime.Surface and underside condition of bridge deck, which is reflected by level 2 rating data, decreases over time for both SF and FRSF concrete bridge deck overlays, especially for SF concrete bridge deck overlay;

6. Overall, fibre has improved the performance of silica fume concrete overlay, especially the crack and corrosion performance.

CHAPTER 6 SUMMARY AND CONCLUSION

6.1 Summary of Research

The main contribution of this research study can be classified in the following four areas:

- This study conducted a literature review on bridge deck testing, modeling and performance evaluation for repairing bridges in Alberta.
- This study compared the performance of five historical and current bridge deck repair methods in Alberta including high performance concrete overlay (HPCO), high density concrete overlay (HDOL), hybrid concrete overlay (HYOL), asphalt concrete with polymer membrane overlay (ACPM) and thin polymer overlay (TPOL).
- Two recent bridge deck repair methods, silica fume and fiber reinforced silica fume concrete overlay, were compared by different performance indicators in this study.
- The impact of two site factors, traffic and condition of bridge at rehabilitation time, were investigated.

Mean service life of each type of bridge deck overlay, as predicted by the AT and ET models of each type, were calculated and compared. The results showed that HDOL had the highest service life, while the results from expert opinion methods indicated that HPCO had the highest service life. One explanation for this difference could be that the HDOL repair method is more sensitive to construction quality. Poor construction HDOL bridges received second repair less than 8 years after the first repair. Therefore, these bad performance HDOL bridges could be categorized into a HYOL group. Using this grouping technique, the average service life of HDOL increased significantly.

Fibers greatly improve the properties of silica fume concrete overlay by decreasing the crack length and crack density of a bridge deck. A small crack will be less likely to allow de-icing salt to penetrate into the reinforcement steel, which results in a longer service life. The rehabilitation service life of FRSF concrete overlay bridges was much longer than that of SF concrete overlay bridges, which also proved the positive effectiveness of fibers.

Traffic and rehabilitation time had a great impact on the bridge deck service life. With higher traffic, the bridge deck service life will decrease greatly. If the bridges were rehabilitated at an earlier time, the service life of a bridge deck overlay will increase significantly with less cost.

6.2 Recommendations for Future Research

Future research should investigate the following points in order to better understand the effectiveness of bridge deck rehabilitation overlay methods:

- There are many bridge deck overlay performance indicators such as total crack length, crack density, debonding performance and condition rating. Analytical Hierarchy Process (AHP) method can be used to select the best rehabilitation method by giving each performance indicator one weight, scoring each rehabilitation method with one value based on its performance in one performance indicator, multiplying the weight value with the score, and summarizing these values together. The rehabilitation method with the highest value indicates that this method has the best overall performance.

- Service life of a bridge deck overlay is one bridge deck overlay performance indicator. It is concluded in this study that the longer the service life, the better effective the bridge deck overlay is. In future studies, life cycle cost data should be taken into consideration because service life and cost ratio will help engineers to choose the most cost-efficient overlay method.

- There are many factors which have an impact on the service life of a bridge deck rehabilitation method such as traffic, rehabilitation time and deicing salt. This study investigated the impact of traffic and rehabilitation time. However, the impact of deicing salt was not considered in this study, due to the fact that there is already a record of the amount of deicing salt applied in each bridge deck module. Laboratory methods are expected to develop in order to consider the impact of de-icing salt on the bridge deck overlay service life.

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APPENDICES

Appendix A: Percentage of deck area with CSE < -300 mV predicted by AT and ET models for bridges tested during year 2000 and 2001

Bridge File	Test Year	Percentage of deck area with CSE < -300 mV		
		Measured	ET	AT
167	2000	34.3	30	56
233	2000	17.9	11	20
272	2000	3.8	10	36
2233	2000	2	12.5	13
2401	2000	17.2	21	56
6809	2000	0.6	0	48
7295	2000	0	60	6
7461	2000	10	12	50
8036	2000	38.6	14	50
8495	2000	86.7	20	70
8719	2000	41.6	15	60
9219W	2000	6	8	11
13824	2000	11.9	9	41
70509	2000	0.3	40	6
72535S	2000	1.9	29.5	40
72810W	2000	1.5	1.5	24
72810E	2000	1.5	1.5	22
73389	2000	0.4	0	62
73420	2000	2.3	15	9
73819	2000	39.1	39.1	2
74137	2000	5.7	33	41
74217	2000	16	7	33
74222	2000	1.2	1.2	4
74232	2000	73.6	10	75
74233	2000	97.5	42	66
74352W	2000	18.6	18	56
74353W	2000	10.6	36	24
74354E	2000	6.5	1.5	14
74426	2000	1	4	9
74540	2000	0	0	4
74678	2000	3.4	13	22
75021	2000	1.2	12	16
75055S	2000	21.7	13	58
75070	2000	0.2	25	32
75187	2000	46.5	31	58
75336	2000	23.3	32	53
75340N	2000	25.1	20	26
75340S	2000	27.6	16	50
75529	2000	0	7	4
75538	2000	18.7	14	7
75722	2000	41.3	18	50
75723	2000	0	0	6

75945	2000	1.8	13	12
76301	2000	2.5	6	30
76382N	2000	23.3	24	70
76392	2000	0.4	0.4	8
76927	2000	1.7	1	13
77073	2000	0	13	6
77091WC	2000	5.7	14	10.5
77129	2000	5.4	12	10
77177	2000	13	4.5	11
77315	2000	5	10	17
77349	2000	2.9	2.5	18
77426	2000	3.5	7	18
77846	2000	0.1	0.1	16
78104	2000	5.3	13	15
78765	2000	0.6	57	16
78808	2000	5.7	62.5	10
78896	2000	1.2	8	8
79325	2000	5.4	6	32
79443	2000	0.1	76	10
79464	2000	1.6	40	12
79671	2000	0.9	0	75
756N	2001	9.4	1.5	17
786	2001	3.7	57	17
875	2001	2	26	2
1085	2001	13.1	20	61
1153	2001	11.9	68	12
1409	2001	8.6	14	9
1980	2001	49.7	12.5	36
2302	2001	17.3	68	4
2431	2001	0.9	14	2
7871	2001	3.8	65	4
8077	2001	1.9	59	6
8303	2001	26.8	14.1	39
8641	2001	0.6	0.6	5
9551	2001	54.3	10	93
9899	2001	38.9	18.5	42
9943	2001	2.3	3.5	4
13742	2001	5.4	30	6
13852	2001	27.3	10	7
70009	2001	10.9	4.5	24
70022	2001	7.1	4	14
70156	2001	1.3	21.5	12
70277	2001	4.7	2.3	10
70594	2001	37.5	25	27
70935	2001	37.9	11	24
71145	2001	14.5	14	16
72467	2001	6.5	8	11
73184	2001	0.6	90	9
73407	2001	62.2	23.5	83

73621	2001	8.1	18	6
74353E	2001	4.4	6	13
74354E	2001	2.6	17	16
74355W	2001	5.6	3.6	15
74355E	2001	15.8	14.6	16
74381	2001	60	40	72
74447	2001	2.8	1.5	17
75058N	2001	79.1	32	93
75058S	2001	70.4	19.5	73
75186	2001	24.9	4.5	22
75193W	2001	30	4	20
75193E	2001	22.1	5	24
75197	2001	21.3	15	11
75332N	2001	20.6	70	68
75332S	2001	44.6	35	54
75335N	2001	2.3	30	20
75335S	2001	1	13	8
75337N	2001	15.3	55	17
75337S	2001	22.9	55	40
75338N	2001	5.4	2	15
75338S	2001	5.2	5.2	13
75383	2001	18.1	3	20
75535N	2001	26.2	7	8
75535S	2001	1	4	75
75651N	2001	13.2	20	22
75744	2001	6.6	22	9
75754	2001	64	30	48
75932	2001	53.9	25	50
76177	2001	1	23	6
76186	2001	0.7	2	12
76615	2001	35.5	22	50
77088	2001	16.5	74	27
77528W	2001	5.9	30	98
77534	2001	23.4	13	21
77556W	2001	0	6	24
77753W	2001	4	50	36
77808W	2001	97.8	25	100
77872	2001	0.7	12.5	10
77878	2001	0.4	0.4	9
78123	2001	5.7	10	8

Appendix B: Service life predicted by AT and ET models for all groups of bridges

Group HPCO

Bridge File Number	Construction Year	Traffic (AADT)	1st Repair		2nd Repair		3rd Repair		T1		T2	
			Type	Year	Type	Year	Type	Year	ET	AT	ET	AT
272	1974	2750	SFSF Con OL	1994					18	14	25	20
2401	1959	320	SFSF Con OL	1993					47	19	42	18
6809	1972	500	SFSF Con OL	1998					-	-	35	19
7461	1960	1340	Curb Sealed	1987	SFSF Con OL	1991			29	21	30	22
13486	1970	870	ACP Membrane	1985	Chip Coat	1994	SFSF Con OL	1998	31	21	28	19
70247	1954	200	Curb Sealed	1988	SF Pyrament Con OL	1989			24	19	17	13
72535S	1958	5630	SFSF Con OL	1999					25	13	31	18
73420	1972	150	SFSF Con OL	1996					23	12	36	20
74137	1958	2070	SFSF Con OL	1998					27	14	32	18
74426	1956	900	SFSF Con OL	1994	Chip Coat	1999			23	8	36	22
75538	1970	170	SFSF Con OL	1994					36	9	45	16
75945	1966	2750	SFSF Con OL	1995					23	11	37	21
76382N	1967	100	SFSF Con OL	1994					35	12	30	10
76392	1973	13060	SFSF Con OL	1995					14	10	25	21
76927	1970	310	SFSF Con OL	1995					21	7	45	21
77091WC	1970	13370	SFSF Con OL	1995					24	14	30	19
77129	1975	500	SFSF Con OL	1995					17	12	27	19
77177	1971	4310	SFSF Con OL	1991	Deck Sealed+Crack Repaired	1999			11	11	21	20
77315	1975	8490	Curb Sealed	1990	FRSF	1995	Chip Coat	1999	25	12	35	20
77426	1975	930	Chip Coat	1994	SFSF Con OL	1998			21	5	37	16
78041N	1976	16820	SFSF Con OL	1998					24	17	25	18
78104	1976	1120	Curb Sealed	1990	SFSF Con OL	1995			23	13	31	19

78896	1968	1450	Curb Sealed	1986	SFSF Con OL	1995			-	-	28	12
1085	1973	930	Steel Fibre Silica Fume OL	1993	Deck Sealed	1998			29	23	25	19
1767	1961	50	Asphalt with Membrane	1994	Steel Fibre Silica Fume Concrete OL	1999			10	6	20	19
7109	1979	150	FRSF OL	1993					-	-	28	22
8303	1960	1910	Curb Sealed	1988	FRSF Con Ol	1992			21	12	21	12
8641	1967	1780	SFSF Con OL	1994					-	-	46	22
9259	1976	630	FRSF OL	1995					28	19	29	20
9333	1998	340	SF Con OL	1998					10	6	26	20
70277	1977	2230	SFSF Con OL	1994	Chip Coat	2000			-	-	32	21
71145	1957	1060	FRSF OL & Cortex inhibitor	1994					18	15	20	18
72467	1963	360	SFSF Con OL	1995					19	12	35	20
75335N	1962	25950	Deck Sealed	1991	SF Con OL	1996	Deck Sealed+Chip Coat	1998 +2000	16	20	16	21
75335S	1962	25950	SF Con OL	1996	Deck Sealed	1998			18	13	25	21
75338N	1962	20870	Chip Coat	1987	SFSF Con OL	1993			-	-	35	21
75338S	1962	20870	Chip Coat	1987	SFSF Con OL	1993			21	12	35	21
75535N	1964	20870	Chip Coat	1986	SFSF Con OL	1996	Chip Coat	1998	-	-	17	18
75651N	1964	18360	Deck Sealed	1991	SFSF Con OL	1995	Chip Coat	2001	-	-	26	18
75651S	1964	18360	Curb Sealed	1991	SF Con OL	1995			18	11	17	11
76186	1967	2380	Deck Sealed	1990	SFSF Con OL	1992			11	13	28	23
76615	1969	120	Deck Sealed	1986	SFSF Con OL	1988			22	17	22	17
77528	1975	8490	SFSF Con OL	1985	SFSF Con OL	1998	Chip Coat	1998	-	-	29	18
77872	1976	6760	Deck Sealed	1990	SFSF Con OL	1992			-	-	34	24
77878	1975	580	SFSF Con OL	1995					-	-	39	22
7836	1960	1910	FRSF Con Ol	1992					28	23	25	19
73779	1975	3260	FRSF Con Ol	1993	Chip Coat	2001			15	11	27	22

74710	1970	760	FRSF Con Ol	1994					13	12	23	19
74969	1960	990	FRSF Con Ol	1992					32	17	37	20
76558	1970	1680	FRSF Con Ol	1994					21	15	28	21
76639	1972	670	FRSF Con Ol	1995	Chip Coat	2003			12	10	28	20
76649W	1969	28310	FRSF Con Ol	1992					16	16	15	17
76650N	1974	24550	FRSF Con Ol	1994					24	15	33	22
76650S	1974	24550	FRSF Con Ol	1994					12	11	21	18
77175	1970	760	FRSF Con Ol	1994					32	21	32	21
710	1963	780	SFSF Con OL	1998					23	12	37	21
1797	1964	990	FRSF Con Ol	2000					-	-	31	20
1894	1962	850	SFSF Con OL	1996					-	-	28	21
2143	1970	8290	Chip Coat	1994	FRSF Con	1997			21	11	29	18
8984	1981	940	SFSF Con OL	1995					-	-	42	22
13625	1968	670	SFSF Con OL	1996					28	14	16	8
71291	1972	570	FRSF Con OL	1996					7	7	23	19
73837W	1976	4030	HDOL	1977	SFSF Con OL	1999			-	-	15	14
73920W	1976	4320	SFSF Con OL	1997					-	-	18	10
74227	1974	880	SFSF Con OL	1992					-	-	20	15
74455	1956	1970	FRSF Con OL	1992	Deck&Curb Sealed	1998			20	18	26	22
75331S	1962		SF Con OL	1996	Deck&Curb Sealed	1998			13	14	16	19
75701	1968	580	FRSF	1995					27	19	30	21
76378	1971	1680	SF Con OL	1997					23	23	22	22
76658	1971	1000	SF Con OL	1996					30	18	33	20
76850	1970	2530	FRSF	1998					11	10	19	19

Group HDOL

Bridge File Number	Construction Year	Traffic (AADT)	1st Repair		2nd Repair		3rd Repair		T1		T2	
			Type	Year	Type	Year	Type	Year	ET	AT	ET	AT
8028	1961	3870	HDOL	1985					13	13	19	17
8036	1959	1050	HDOL	1981					25	25	31	31
73389	1985	1900	HDOL	1985					-	-	34	28
74217	1954	1580	HDOL	1979	Polymer OL	1991			20	13	25	16
74678	1958	2850	HDOL	1978	Epoxy OL	1992	Chip Coat	2000	27	36	25	32
78595	1980	19920	HDOL	1980	Chip Coat	1983	Chip Coat	1995	-	-	27	31
79443	1979	6180	HDOL	1979	Chip Coat	1984			24	20	34	28
887	1957	1930	HDOL	1984	Deck Sealed	1996			19	19	29	26
9551	1951	1630	HDOL	1982	Polymer OL	1992	Chip Coat	1993	30	22	14	14
75058N	1961	19650	HDOL	1980	Polymer OL	1989	Chip Coat	1994	34	24	18	18
75058S	1961	19650	HDOL	1980	Polymer OL	1989	Chip Coat	1995	24	24	19	19
75193E	1961	24900	HDOL	1985	Deck Sealed	1990	Epoxy, Chip Coat	1993	30	23	25	20
75193W	1961	24900	HDOL	1985	Deck Sealed	1990	Epoxy, Chip Coat	1993	25	24	20	19
77088	1979	6180	HDOL	1979	Chip Coat	1985	Deck Sealed	1987	19	18	23	22
9910	1958	2450	HDOL	1981	Polymer OL	1989			30	29	25	24
76652	1971	24550	HDOL	1982	Epoxy OL	1993	Chip Coat	1996	25	24	22	22
903	1954	1190	HDOL	1982					26	26	29	30
904	1954	1190	HDOL	1983					25	23	33	30
2235	1957	1170	HDOL	1979	Chip coat	1995			30	32	29	30
13166	1935	280	HDOL	1977					-	-	25	25
73426	1959	1050	HDOL	1981	Polymer OL	1989			36	36	32	31
74229	1975	630	HDOL	1977	Polymer OL	1997			31	32	30	31

Group HYOL

Bridge File Number	Construction Year	Traffic (AADT)	1st Repair		2nd Repair		3rd Repair		T1		T2	
			Type	Year	Type	Year	Type	Year	ET	AT	ET	AT
233	1960	2020	HDOL	1982	Polymer OL	1989			34	28	29	24
8495	1958	1350	HDOL	1986	Deck Sealed	1988	Poly OL	1992	16	16	6	6
8719	1954	1860	HDOL	1985	Epoxy OL	1991	Chip Coat	1997	25	22	18	17
74352W	1957	14430	HDOL	1984	Polymer OL	1990	Chip Coat	1995	25	25	22	22
74353W	1958	14430	HDOL	1983	Epoxy OL	1987	Chip Coat	1995	24	29	23	25
70935	1963	1530	HDOL	1984	Polymer OL	1990	Chip Coat	1999	23	22	20	19
73407	1960	520	HDOL	1985	Polymer OL	1989	Chip Coat	1998	26	16	6	6
74353E	1958	14430	HDOL	1985	Epoxy OL	1989	Chip Coat	1995	27	26	28	27
74354W	1970	15400	HDOL	1982	Polymer OL	1988	Chip Coat	1995	32	28	30	27
75186	1960	3660	HDOL	1986	Polymer OL	1990	Chip Coat	1998	16	16	25	19
75932	1966	4750	HDOL	1984	Epoxy OL	1990	Chip Coat	1994	21	23	18	18
75555	1963	2860	HDOL	1985	Epoxy OL	1989	Chip Coat	1993	35	23	25	19
75677	1964	500	HDOL	1982	Polymer OL	1988	Chip Coat	1994	58	25	33	21
2010	1982	1500	HDOL	1990	Polymer OL	1997	Con Overlay	1997	28	30	24	24
74228	1972	2200	HDOL	1982	Polymer OL	1986	Chip Coat	1996	22	22	20	20
75051N	1960	6720	HDOL	1981	Polymer OL	1988	Chip Coat	1993	25	24	23	23
75051S	1960	6720	HDOL	1981	Polymer OL	1988	Chip Coat	1993	25	24	24	24
75111	1961	1100	HDOL	1986	Polymer OL	1989	Chip Coat	1995	40	28	34	25
75195E	1961	20220	HDOL	1982	Polymer OL	1987	Chip Coat	1994	24	25	20	20
75195W	1961	20220	HDOL	1982	Polymer OL	1987	Chip Coat	1994	25	24	24	23

Group ACPM

Bridge File Number	Construction Year	Traffic (AADT)	1st Repair		2nd Repair		3rd Repair		T1		T2	
			Type	Year	Type	Year	Type	Year	ET	AT	ET	AT
74222	-	1600	Asphalt and Membrane	1989	Girders Painted	1991	Chip Coat	1998	-	-	21	33
75722	1964	100	Membrance ACP	1992					16	12	13	11
9899W	1972	4820	Curb Sealed	1990	ACP & Membrane	1991	ACP replaced	2000	6	6	13	14
75197	1961	60	Curb Sealed	1989	ACP & Membrane	1990	Chip Coat	1992	12	12	17	18
75337N	1962	20870	ACP & Membrane	1989	Chip Coat	1994			18	22	18	22
75337S	1962	20870	ACP & Membrane	1989	Chip Coat	1994			17	18	17	19
75744	1965	6180	Asphalt with Membrane	1987	ACP	1991			14	18	16	25
457	1959	2220	ACP & Membrane	1998	Chip Coat	1998			11	8	19	18
13587	1965	1200	ACP & Membrane	1990	Chip Coat	1991			16	15	18	18
73922	1975	520	ACP & Membrane	1989					8	8	12	12
74978E	1960	8720	ACP & Membrane	1988	Chip Coat	1998			17	16	19	18
75339S	1962	20870	Membrane ACP	1988	Chip Coat	1994			20	18	21	18
76660	1970	1640	ACP & Membrane	1989					19	18	20	20
189	1974	1490	ACP & Membrane	1989	Chip Coat	1991			9	9	17	16
74352E	1968	14430	ACP & Membrane	1988	Chip Coat	1992			16	16	21	19
75919S	1967	5630	ACP & Membrane	1988					-	-	18	18
77521	1975	7900	ACP & Membrane	1988	Chip Coat	1994	SFSF	1998	20	17	21	18

Group TPOL

Bridge File Number	Construction Year	Traffic (AADT)	1st Repair		2nd Repair		3rd Repair		T1		T2	
			Type	Year	Type	Year	Type	Year	ET	AT	ET	AT
167	1965	380	Deck Sealed	1987	Polymer OL	1992	Chip Coat	1999	20	16	17	12
611	1963	200	Polymer OL	1985	Chip Coat	1997			13	13	14	14
75055S	1960	45810	HDOL	1980	Deck Sealed+Chip Coat+Curb Sealed	1986 +1988+ 1991	Poly OL	1996	20	7	22	8
75336	1961	4910	Polymer OL	1990	Chip Coat	1994			18	17	18	16
76301	1977	4110	HDOL	1977	Deck Sealed+Chip Coat	1986+ 1994	Poly OL	1997	9	7	12	13
77198	1970		PMA OL	1990					-	-	15	27
77349	1978	1710	HDOL	1982	Chip Coat	1993 +1995- 1997	Poly OL	1997	-	-	17	15
79325	1984	2570	Epoxy OL	1998					10	9	21	22
962	1962	500	Epoxy OL	1988					-	-	23	33
1153	1959	1720	Polymer OL & Chip Coat	1998					-	-	11	15
1409	1957	2220	Polymer OL	1993	Chip Coat	1997			9	9	13	21
1741	1961	1900	Deck Sealed	1987	Polymer OL	1992	Chip Coat	1995	12	15	22	24
1980	1962	5010	Polymer OL	1984	Polymer OL	1993	Chip Coat	1998	8	8	9	9
2430	1963	2750	HDOL	1981	Deck Sealed	1986	Epoxy, Chip Coat	1992	12	11	23	17
70594	1954	1530	HDOL	1981	Polymer OL	1993			14	16	11	11
74381	1964	4060	HDOL	1980	Polymer OL	1991	Chip Coat	1995	17	16	16	16
1227	1964	960	Epoxy OL	1988	Chip Coat	1993			-	-	22	20

71316	1962	1080	Widened	1984	Polymer OL	1988	Chip Coat	1997	16	16	20	18
76034	1966	2180	Polymer OL	1990	Chip Coat	1993			16	16	15	14
977	1978	4390	Polymer OL	1997					12	14	20	24
72094		1530	HDOL	1981	Polymer OL	1991	Chip Coat	1998	22	17	18	15
73425	1959	1170	Latex Mod. Con OL	1987	Polymer OL	1991	Chip Coat	1998	27	21	20	16
73429	1967	810	Polymer OL	1985					37	29	31	25
73919E	1955	4820	HDOL	1985	Polymer OL	1993			28	26	23	21
75305	1961	1190	Latex	1984	Polymer OL	1992	Chip Coat	2000	19	15	17	14

Appendix C: questionnaire

Service Life of Alternative Deck Repair Method

Background: Paul Carter and Hamid Soleymani (University of Alberta Civil Engineering Dept) are working on a project for Alberta Transportation. One task involves assessing the cost effectiveness of various Alberta bridge deck repair methods. They are seeking the expert opinion of experienced people on typical repair performance, and request your opinion regarding the repair systems below. Feel free to leave any items blank where you have no opinion, and call Paul at (780)409-9298 if you need clarification of anything.

Instructions: This survey is focused on your experience with deck 'repair' systems, not new construction. So, high density overlay or membrane/ACP applied to new decks is not to be considered. For purposes of this survey, service life is defined as the time until replacement of the overlay system is required, usually due to rideability problems. Most overlays have not yet failed, but use your best judgement as to how long they will last in your opinion in several types of highway situations.

High density overlay is defined as Iowa method low slump, special screed, site-batched, generally placed between 1977 and 1985. Most AT high density overlays are now covered with epoxy/chip coat. Of these, many received epoxy less than 10 years after the high density was done. Since this makes it different to separate the life of the high density versus the life of the two repair methods, we've defined a separate category for hybrids, high density with epoxy placed within 10 years).

Estimated Average Service Life of Deck Repair Systems (yrs)

	Severity of Exposure Conditions		
Deck Repair System	Worst Case (ADT > 5,000)	Typical Case (ADT 1,000 – 5,000)	Best Case (ADT < 1,000)
High density concrete overlay (HD)			
Class FRSF concrete overlay			
Thin epoxy overlay			
Epoxy membrane and asphalt			

Call Paul Carter at 780.409.9298 if you have questions.

Appendix D: performance data for SF and FRSF bridge deck overlays

Group SF

Number of Bridges	Br File	AADT	Clear Deck Area	Year Const.	Year Overlaid	Year Last Tested /Inspected	Deck CSE % > 300 (mV) @ Most Recent Test	Deck CSE % > 300 (mV) @ Time of Rehab	Predicted T2 Value	Chain Drag - % Debonded	Patched/ spalled %	Top M/W cracks (lin m)	Crack Length / Deck Area	W/S - Level 2 Rating	Deck Underside Level 2 Rating
1	01053	510	258	1958	1997	2000	5.9	9.1	Blank	3.5%	0.0%	46	0.18	5.5	8.0
2	01766	1488	420	1967	1998	2005	8.0	Blank	Blank	1.4%	0.0%	82	0.20	7.5	5.5
3	01894	1380	482	1962	1996	2004	3.9	6.0	28	1.2%	0.1%	73	0.15	5.5	5.5
4	02010	1500	622	1956	1997	2004	16.7	Blank	Blank	0.0%	2.0%	243	0.39	7.9	7.9
5	08984	987	373	1981	1995	2004	2.1	Blank	Blank	0.0%	0.0%	94	0.25	8	N
6	71313	770	1047	1960	1999	2005	48.8	Blank	Blank	0.1%	0.1%	108	0.10	7.5	5.5
7	73837	4230	981	1977	1999	2000	18.5	4.0	15	0.0%	0.0%	204	0.21	6.8	8.0
8	73920	2908	812	1976	1997	2000	28.0	22.0	18	0.0%	0.0%	237	0.29	7.0	N
9	75331	6000	532	1962	1996	2000	14.1	34.0	16	0.6%	0.0%	282	0.53	5.1	N
10	75332	25950	688	1962	1997	2002	20.6	86.0	10	0.0%	0.0%	35	0.05	8.0	5.5
11	75332	25950	725	1962	1995	2002	44.6	66.0	9	0.0%	0.0%	73	0.10	5.5	5.5
12	75335	25130	2186	1962	1996	2002	2.3	50.0	16	0.0%	0.0%	614	0.28	7.5	8.0
13	75335	25950	2186	1962	1996	2002	1.0	16.0	25	0.0%	0.0%	556	0.25	5.5	8.0
14	75340	21666	939	1962	1995	2005	31.4	Blank	Blank	0.0%	0.0%	417	0.44	5.5	5.5
15	75340	21666	939	1962	1995	2005	32.8	Blank	Blank	0.0%	0.0%	341	0.36	5.5	5.5
16	75535	20870	590	1964	1996	2002	26.2	8.0	17	2.0%	0.0%	125	0.21	4.0	7.9
17	75535	20870	590	1964	1996	2002	1.0	3.0	12	1.0%	0.0%	266	0.45	4.0	7.9
18	75651	18360	833	1964	1995	2002	13.2	18.0	26	0.0%	0.0%	45	0.05	6.8	4.8
19	75651	18360	833	1964	1995	2002	23.2	30.0	17	0.0%	0.0%	112	0.13	5.5	5.4
20	75816	1112	463	1976	1997	2000	N	N	N	0.0%	0.0%	30	0.06	8.0	N
21	76092	1000	666	1968	1998	2004	28.1	Blank	Blank	0.0%	0.0%	169	0.25	5.5	5.5
22	76128	1540	469	1968	1996	2000	N	N	N	2.1%	0.0%	123	0.26	7.4	N
23	76181	8306	1042	1967	1995	2005	0.5	Blank	Blank	2.2%	0.0%	251	0.24	5.4	5.5
24	76181	8306	1042	1967	1995	2005	1.4	Blank	Blank	3.0%	0.0%	344	0.33	5.5	5.5

25	76382	250	667	1967	1994	2005	10.0	Blank	Blank	0.4%	0.0%	58	0.09	5.5	5.4
26	76646	32190	653	1968	1997	2000	N	N	N	0.9%	0.0%	180	0.28	6.8	N
27	76646	32190	653	1968	1997	2000	N	N	N	0.9%	0.0%	246	0.38	5.5	N
28	76913	1220	526	1974	1997	2001	N	N	N	0.0%	0.0%	112	0.21	8.0	N
29	77471	1800	469	1975	1997	2000	N	N	N	0.0%	0.0%	144	0.31	8.0	N
30	79564	120	1760	1978	1996	1999	N	N	N	0.3%	0.0%	586	0.33	7.0	8.0

Group FRSF

Num.	Bridge File #	AADT	Clear Deck Area	Year Const.	Year Overlaid	Year Last Tested /Inspected	Deck CSE % > 300 (mV) @ Most Recent Test	Deck CSE % > 300 (mV) @ Time of Rehab	Predicted T2 Value	Chain Drag - % Debonded	Patched/ spalled %	Top M/W cracks (lin m)	Crack Length/ Deck Area	W/S - Level 2 Rating	Deck Underside Level 2 Rating
1	00149	620	466	1970	1996	1997	0.0	Blank	Blank	2.1%	0.0%	9	0.02	8.0	7.8
2	00272	2240	1244	1974	1994	2001	3.8	20	25	1.0%	2.0%			5.5	5.4
3	00710	849	499	1963	1998	2004	0.7	16	37	0.0%	0.0%	12	0.02	5.5	8.0
4	00983	1170	337	1959	1998	2003	2.8	Blank	Blank	0.2%	0.0%	0	0.00	8.0	8.0
5	01062	640	577	1960	1997	2000	0.4	Blank	Blank	0.0%	Blank	Blank	Blank	Blank	Blank
6	01074	19984	4641	1966	1998	2005	4.3	Blank	Blank	1.4%	2.0%	52	0.01	8.0	6.4
7	01085	915	1420	1973	1993	2002	13.1	60	25	0.0%	0.0%	27	0.02	7.8	8.0
8	01145	16350	1056	1969	1995	2002	25.2	90	9	0.0%	1.0%	68	0.06	7.6	8.0
9	01664	842	660	1969	1994	1998	N	N	N		0.0%	0	0.00	7.0	8.0
10	01767	50	570	1961	1999	2002	0.6	4	25	0.0%	0.0%	0	0.00	8.0	8.0
11	01797	983	730	1964	2000	2000	0.1	0	31	0.0%	0.0%	0	0.00	8.0	8.0
12	01810	6057	201	1955	2001	2005	0.3	Blank	Blank	0.0%	0.0%	0	0.00	8.0	5.5
13	01916	1035	360	1957	1994	1997	0.1	Blank	Blank	0.6%	0.0%	7	0.02	5.5	5.5
14	02008	500	313	1959	1990	1996	32.6	Blank	Blank	1.6%	0.0%	0	0.00	5.5	6.3
15	02047	5410	522	1966	1993	1997	N	N	N		0.0%	26	0.05	5.5	5.1

16	02143	8270	880	1970	1997	2004	11.8	24	29	0.0%	0.0%	5	0.01	7.8	5.5
17	02401	323	376	1959	1993	2001	17.2	56	42	0.8%	0.0%	14	0.04	8.0	8.0
18	06809	1160	291	1972	1998	2001	0.6	30	35	0.0%	Blank	Blank	Blank	8.0	5.5
19	07109	150	347	1979	1993	2002	3.7	1	28	1.0%	0.0%	2	0.01	8.0	8.0
20	07461	1392	2041	1960	1991	2005	6.8	Blank	Blank	0.0%	0.0%	57	0.03	8.0	8.0
21	07492	1912	894	1972	1995	2003	N	N	N	0.8%	0.0%	38	0.04	7.9	5.5
22	07836	1910	360	1960	1992	1999	43.8	70	25	0.0%	0.0%	19	0.05	5.5	6.8
23	08303	1820	500	1960	1992	2002	26.8	50	21	0.0%	0.0%	13	0.03	7.8	5.5
24	08641	1803	458	1967	1994	2002	0.6	2	46		0.0%	31	0.07	5.5	8.0
25	09099	560	253	1959	1995	2004	6.1	28	44	4.0%	0.0%	0	0.00	8.0	7.5
26	09204	748	333	1956	1999	2001	N	N	N	0.0%	0.2%	76	0.23	5.5	5.5
27	09259	637	622	1976	1995	2002	5.9	36	29	0.0%	0.0%		0.00	5.2	5.5
28	09333	329	1170	1969	1998	2002	1.1	1	26	0.0%	0.0%	56	0.05	5.2	8.0
29	13149	3319	399	1957	1992	1997	N	N	N	0.0%	0.0%	19	0.05	5.5	5.5
30	13181	1624	415	1959	1994	1999	73.2	84	Blank	0.0%	0.0%	34	0.08	6.8	8.0
31	13486	903	444	1970	1998	2001	2.3	60	28	0.0%	0.0%	76	0.17	7.8	7.5
32	13625	640	375	1968	1996	2000	48.7	90	16	0.8%	0.0%	0	0.00	8.0	5.5
33	13821	3032	680	1972	1994	1999	N	N	N	0.4%	0.0%	20	0.03	6.8	N
34	70277	2170	987	1977	1994	2002	4.7	3	32	1.0%	3.3%	N	Blank	5.5	5.4
35	70580	1750	1600	1969	1986	1995	95.3	Blank	Blank	Blank	Blank	Blank	Blank	Blank	Blank
36	70626	1635	906	1965	1992	1999	N	N	N	1.5%	0.0%	325	0.36	5.5	N
37	71054	1047	223	1959	1997	2000	7.9	Blank	Blank	0.0%	0.0%	0	0.00	8.0	8.0
38	71081	590	576	1975	1999	2002	N	N	N	0.0%	0.0%	0	0.00	8.0	8.0
39	71145	1035	2227	1957	1994	2002	14.5	30	20	4.1%	0.0%	185	0.08	7.5	8.0
40	71265	1688	461	1972	2000	2004	N	N	N	0.0%	0.0%	21	0.05	8.0	8.0
41	71291	610	2385	1972	1996	2000	10.2	8	23	0.0%	0.0%	754	0.32	5.5	5.5
42	72467	366	1451	1964	1995	2002	6.5	14	35	2.0%	0.0%	164	0.11	5.5	8.0
43	72533	5626	1103	1964	2001	2005	0.4	Blank	Blank	0.1%	0.0%	89	0.08	5.5	5.5
44	72535	5626	490	1958	1999	2005	0.2	Blank	Blank	0.0%	0.0%	1	0.00	5.5	5.5
45	72631	830	597	1970	1996	2000	84.0	34	5	0.0%	0.0%	0	0.00	5.5	5.1
46	73277	1311	1424	1975	1996	2005	16.2	Blank	Blank	0.6%	0.0%	78	0.05	8.0	5.4

47	73420	170	255	1972	1994	2001	2.3	14	36	0.0%	0.0%	78	0.31	5.5	5.4
48	73779	3032	1005	1975	1993	1999	4.6	1	27	0.0%	0.0%	37	0.04	6.4	8.0
49	74137	2222	466	1975	1998	2005	5.2	26	32	4.1%	0.0%	72	0.15	7.8	5.5
50	74227	778	4366	1974	1992	2000	26.1	20	20	2.8%	0.0%	426	0.10	5.5	5.5
51	74232	1404	2197	1966	1993	2005	90.4	30	6	1.3%	1.0%	514	0.23	5.5	5.5
52	74236	970	2673	1970	1992	1999	8.5	2	16	0.0%	0.0%	590	0.22	4.9	4.8
53	74426	850	468	1956	1993	2001	1.0	6	36		0.0%		0.00	3.0	7.8
54	74455	1990	776	1956	1992	2000	5.9	26	26	0.9%	0.0%	39	0.05	5.4	5.4
55	74596	100	639	1966	1994	1999	90.0	Blank	Blank	0.0%	0.0%	74	0.12	7.4	7.5
56	74600	16193	429	1971	1988	1999	N	Blank	Blank	0.0%	Blank	N	Blank	6.8	N
57	74710	760	476	1970	1994	1999	5.2	15	23	0.0%	0.0%	60	0.13	5.1	7.5
58	74969	1050	617	1960	1992	1999	9.8	30	37	0.0%	0.0%	0	0.00	8.0	8.0
59	75054	1677	1744	1967	1994	1999	3.1	90	37	1.1%	0.0%	650	0.37	5.3	5.4
60	75187	766	2774	1961	1993	2005	23.2	80	9	2.3%	0.0%	243	0.09	5.5	7.8
61	75338	21666	1078	1962	1993	2002	5.4	5	35	0.0%	0.0%	4	0.00	5.5	8.0
62	75338	21666	1171	1962	1993	2002	5.2	15	35	2.0%	0.1%	2	0.00	8.0	7.9
63	75383	2364	597	1963	1994	2002	18.1	6	17	3.0%	0.0%	28	0.05	7.3	8.0
64	75538	201	934	1970	1994	2005	38.3	Blank	Blank	0.0%	0.0%	0	0.00	5.5	7.8
65	75539	1620	525	1963	1998	1998	0.4	Blank	Blank	0.0%	0.0%	0	0.00	8.0	5.5
66	75543	32490	1011	1965	1994	2001	N	N	N	3.0%	0.0%	161	0.16	5.5	5.4
67	75543	32480	1011	1965	1994	2001	N	N	N	1.0%	0.1%	128	0.13	5.5	5.4
68	75623	21666	407	1963	1987	2001	N	N	N		0.0%	142	0.35	5.4	5.4
69	75694	270	427	1975	1998	2005	3.4	Blank	Blank	0.0%	0.0%	0	0.00	8.0	8.0
70	75701	597	1867	1968	1995	2004	7.1	40	30	0.3%	0.0%	30	0.02	7.9	5.5
71	75754	50	842	1964	1994	2002	64.0	42	7	1.0%	0.0%	59	0.07	7.5	7.9
72	75760	12235	1759	1967	1992	1999	7.3	Blank	Blank	0.0%	0.0%	90	0.05	5.5	5.2
73	75931	400	716	1966	1992	1999	83.8	95	Blank	0.0%	0.0%		0.00	4.2	5.5
74	75945	2240	875	1966	1995	2001	1.8	16	37	0.0%	0.0%	14	0.02	8.0	5.5
75	76056	2648	781	1968	1993	2000	N	N	N	0.0%	N	N	Blank	8.0	N
76	76059	2648	734	1968	1993	2000	N	N	N	0.0%	N	N	Blank	5.4	N
77	76063	1190	609	1969	1997	2000	N	N	N	0.0%	0.0%	0	0.00	7.8	N

78	76094	10000	973	1968	1999	2000	N	N	N	0.4%	0.0%	0	0.00	8.0	8.0
79	76186	2600	718	1967	1992	2002	0.7	5	28	0.0%	0.0%	296	0.41	5.5	5.5
80	76212	360	748	1968	1992	1999	5.5	14	49	0.3%	0.0%	0	0.00	5.5	8.0
81	76223	990	458	1966	1996	2001	N	N	N	0.0%	0.0%	0	0.00	8.0	8.0
82	76339	32190	1337	1969	1996	1998	N	N	N	0.0%	0.0%	0	0.00	8.0	7.8
83	76339	32190	1337	1969	1996	1998	N	N	N	0.0%	0.0%	0	0.00	8.0	7.8
84	76378	1750	770	1971	1997	2000	1.0	56	22	0.0%	0.0%	0	0.00	5.5	5.5
85	76381	2682	991	1967	1992	1999	10.0	50	14	0.2%	0.0%		0.00	5.4	5.4
86	76392	16350	1135	1973	1995	2001	0.4	10	25	0.0%	0.0%	367	0.32	5.5	5.5
87	76528	830	364	1969	1996	1997	N	N	N	0.0%	0.0%	5	0.01	8.0	N
88	76540	1395	720	1968	2003	2005	4.1	Blank	Blank	0.0%	0.0%	0	0.00	8.0	N
89	76558	1750	581	1970	1994	1999	5.7	Blank	28	1.2%	0.0%	75	0.13	6.5	5.4
90	76566	100	666	1970	1996	1997	N	Blank	N	0.0%	0.0%	4	0.01	8.0	5.1
91	76639	640	241	1972	1995	1998	1.6	Blank	28	0.0%	0.0%	11	0.05	5.5	8.0
92	76648	10000	989	1969	1993	1999	33.2	Blank	11	0.1%	10.1%	85	0.09	5.5	5.5
93	76649	32190	438	1969	1992	1999	38.3	Blank	15	3.4%	0.0%	13	0.03	5.5	4.0
94	76650	26780	1302	1974	1994	1999	8.0	18	33	0.2%	0.0%	250	0.19	5.4	7.8
95	76650	26780	1469	1974	1992	1999	10.8	16	21	0.2%	0.0%	15	0.01	7.3	7.8
96	76658	1000	570	1971	1996	2004	6.3	36	33	0.0%	0.0%	39	0.07	7.3	5.5
97	76669	830	402	1968	1994	1997	N	N	N	0.8%	0.0%	39	0.10	5.4	N
98	76848	2930	989	1969	1989	2004	34.3	90	17	0.2%	N	N	N	5.5	5.5
99	76850	2194	666	1970	1998	2004	9.1	10	19	0.0%	0.0%		0.00	5.5	5.5
100	76927	270	298	1970	1995	2001	1.7	4	45	2.0%	0.0%	102	0.34	5.4	5.5
101	77091	14080	595	1970	1997	2003	1.2	22	30	0.0%	0.0%	70	0.12	6.5	8.0
102	77091	14080	595	1970	1997	2003	2.6	Blank	Blank	0.3%	0.0%	35	0.06	8.0	8.0
103	77091	14080	351	1970	1995	2001	5.7	Blank	Blank	1.4%	1.0%	112	0.32	5.4	5.5
104	77126	6685	1471	1972	1991	2000	7.7	16	43	3.6%	3.7%	136	0.09	7.8	8.0
105	77129	484	622	1975	1995	2001	5.4	20	27	0.0%	0.0%	150	0.24	5.5	5.5
106	77175	760	254	1970	1994	2003	5.0	40	32	2.4%	0.0%	44	0.17	6.5	7.5
107	77177	4448	2044	1971	1991	2005	6.8	10	21	1.9%	0.0%	11	0.01	7.9	5.5
108	77212	500	459	1970	1999	2001	N	N	N	1.0%	0.0%	3	0.01	8.0	N

109	77254	3742	1513	1975	1994	2004	27.7	25	25	0.2%	0.0%	403	0.27	5.5	7.4
110	77315	7181	995	1975	1995	2001	5.0	18	35	3.0%	N	N	Blank	5.5	8.0
111	77426	955	739	1975	1998	2001	3.5	4	37	0.0%	Blank	16	0.02	Blank	Blank
112	77466	5597	537	1973	1993	1997	N	N	N	0.0%	0.0%	17	0.03	8.0	N
113	77486	1613	319	1975	1994	2001	0.1	Blank	Blank	N	Blank	Blank	Blank	8.0	5.5
114	77487	1613	241	1974	1992	1998	N	N	N	N	Blank	Blank	Blank	Blank	Blank
115	77503	113	482	1974	1992	2005	2.0	Blank	Blank	0.0%	0.0%	2	0.00	5.5	8.0
116	77521	9050	1818	1977	1998	2000	9.5	45	21	0.3%	0.0%	4	0.00	8.0	5.5
117	77528	7181	915	1975	1985	2002	5.9	Blank	Blank	0.0%	N	N	N	N	7.8
118	77847	2918	490	1975	1997	1998	N	N	N	0.0%	0.0%	0	0.00	8.0	N
119	77872	7064	736	1976	1992	2002	0.7	40	34	1.3%	0.0%	250	0.34	4.0	8.0
120	77878	738	644	1975	1995	2002	0.4	0	39	0.0%	0.0%	7	0.01	8.0	5.5
121	78041	18614	4015	1976	1998	2005	7.3	40	25	0.0%	N	165	0.04	7.5	N
122	78104	1112	1394	1976	1995	2001	5.3	12	31	2.0%	0.0%	148	0.11	5.4	5.5
123	78220	300	201	1973	1997	2001	N	Blank	Blank	0.0%	0.0%	0	0.00	8.0	N
124	78896	1395	1059	1968	1995	2001	1.2	6	28	0.0%	0.0%	4	0.00	8.0	8.0